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TRANSACTIONS.

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535.

(Vol. XXVII.—July, 1892.)

THE WEEHAWKEN ELEVATORS AND VIADUCT.

By THOMAS E. BROWN, Jr., M. Am. Soc. C. E., and
GEORGE H. BLAKELEY, Jun. Am. Soc. C. E.

GENERAL DESCRIPTION.

By GEORGE H. BLAKELEY, Jun. Am. Soc. C. E.

This structure was constructed as a part of a general plan to provide rapid transit for the northern part of Hudson County, N. J., by the North Hudson County Railway Company, which owns and operates all the surface and elevated lines of railway north of Pavonia Avenue in Jersey City. It is situated at Weehawken, opposite the foot of West 49th Street, New York City, on the lands of the New York, West Shore and Buffalo Railroad Company, and is reached by ferries from the foot of Jay Street and West 42d Street, New York.

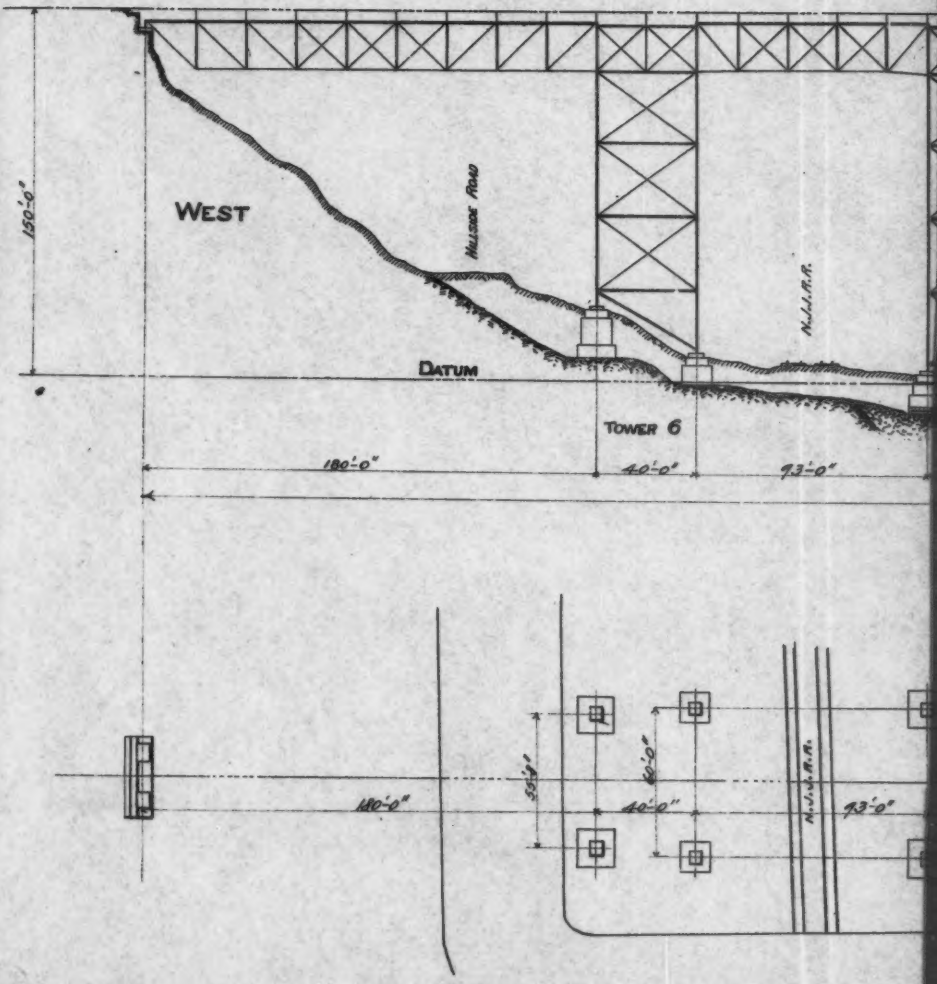
The structure is designed for carrying a double track standard gauge railroad to connect the surface and horse-car lines on the summit of the Palisades with the West Shore ferries, and provide easy and rapid communication with the City of New York. The general plan was

devised to meet the requirements of the situation by the late Mr. J. H. Bonn, President of the North Hudson County Railway Company. Several plans for the structure had been considered before the final plan upon which it is now built was adopted. The first plan considered was for the construction of a tower, located at the base of the Palisades, provided with elevators of sufficient capacity for lifting ordinary light passenger railway coaches from the foot of the bluff to the top of the tower, from which point a short, level span would connect with the summit of the hill. An inclined rack railway was also discussed. After consideration of the subject, the railroad company adopted the present plan, in which the railroad runs out from the level of the top of the Palisades to a terminal station situated nearly over the ferry house of the West Shore Railroad, and elevators provided for lifting and lowering the passengers between the ferry house and the level of the elevated railroad above (Plates I–II).

Preliminary surveys and borings for the foundations, and plans and approximate estimates of cost, were made in the fall of 1889. Financial arrangements having been completed, the construction was begun in the fall of 1890.

The structure as first designed was proportioned for carrying traffic of the same magnitude and with the same motive power as that in use on the elevated railways in New York, namely, 21-ton rapid-transit locomotives on each track. But the consideration of the locality, and the possibility of the development of greater suburban traffic in the future, led the writer to advocate the proportioning of the structure for much heavier loads. For several years a charter has been in existence for the construction of an inclined railway up the face of the Palisades, to reach the desirable residence locality situated on the top of the bluff on the western shore of the Hudson River, but no efforts have been made to carry out construction under this charter. The Palisades at this place rise abruptly, a short distance west of the river, with a slope so steep as to preclude easy access to the summit except by the construction of steep roadways, undesirable for ordinary wagon traffic.

The summit of the Palisades at this point is a plateau about a mile in width which extends to a point nearly opposite Yonkers, with an elevation varying from 200 to 350 feet above the Hudson River. The western slope is a gradual one, and along its base runs the


150'-0"
WEST
HILLSIDE ROAD
N.J.J.R.R.
DATUM
TOWER 6
180'-0"
40'-0"
93'-0"
110'-0"
35'-0"
60'-0"
40'-0"
N.J.J.R.R.
93'-0"

Weehawken Viaduct
General Elevation & Plan

Scale

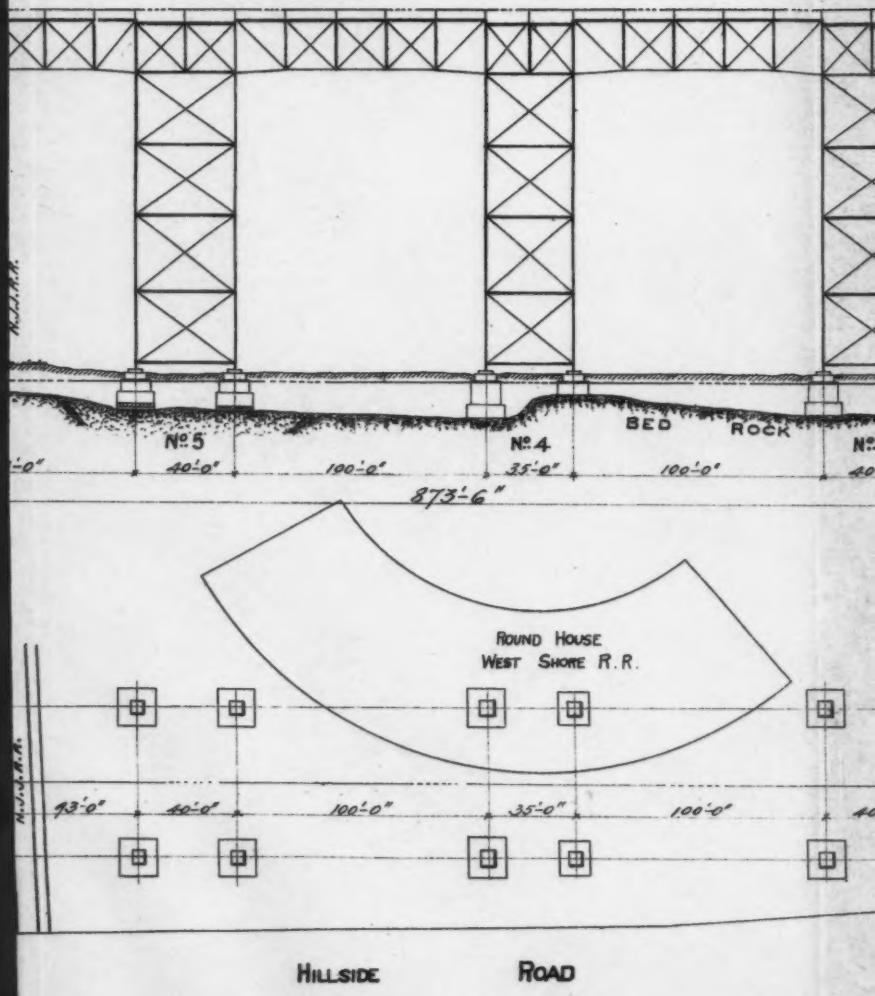


PLATE I
TRANS. AM. SOC. CIV. ENGRS
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BLAKELEY ON
WEEHAWKEN VIADUCT.

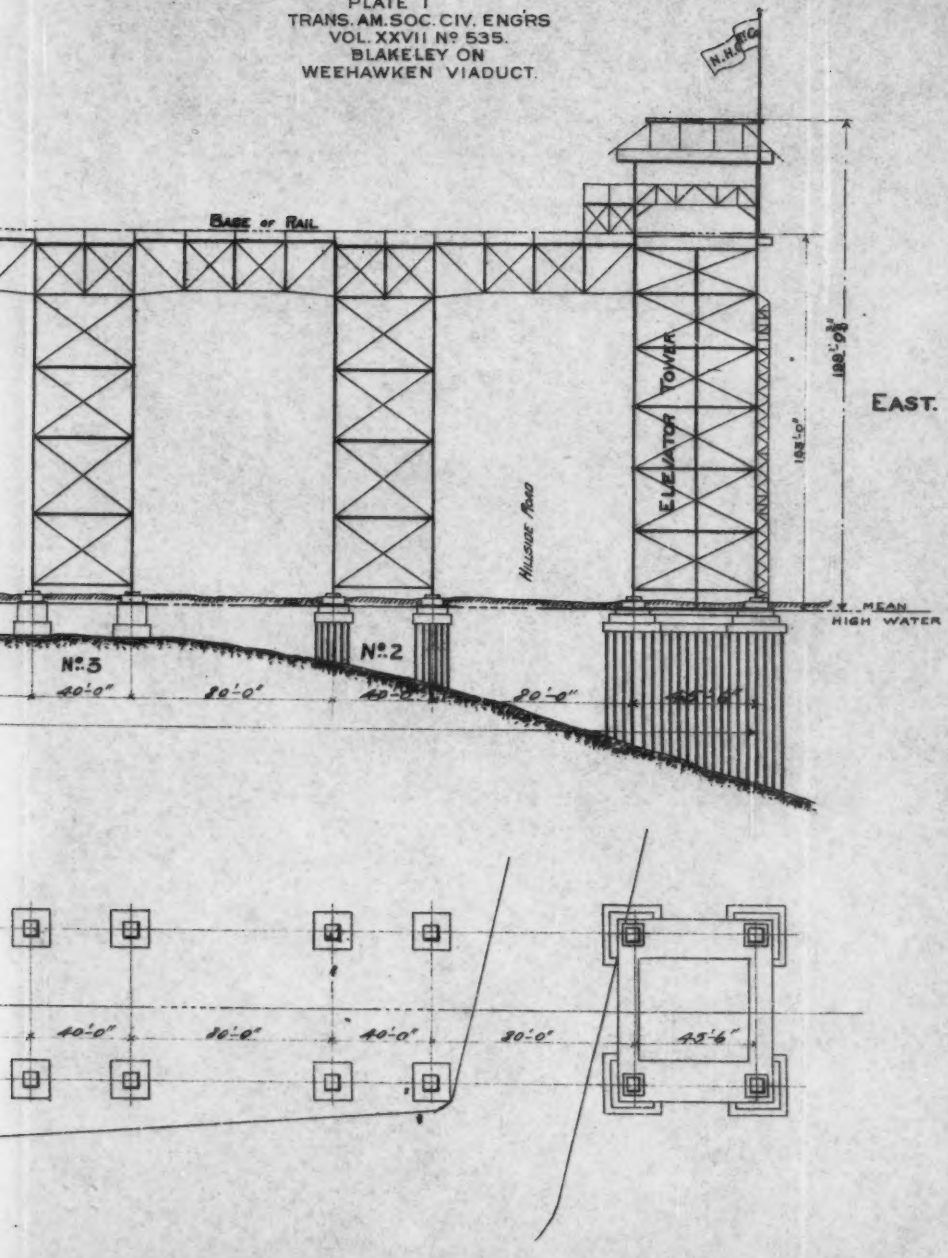
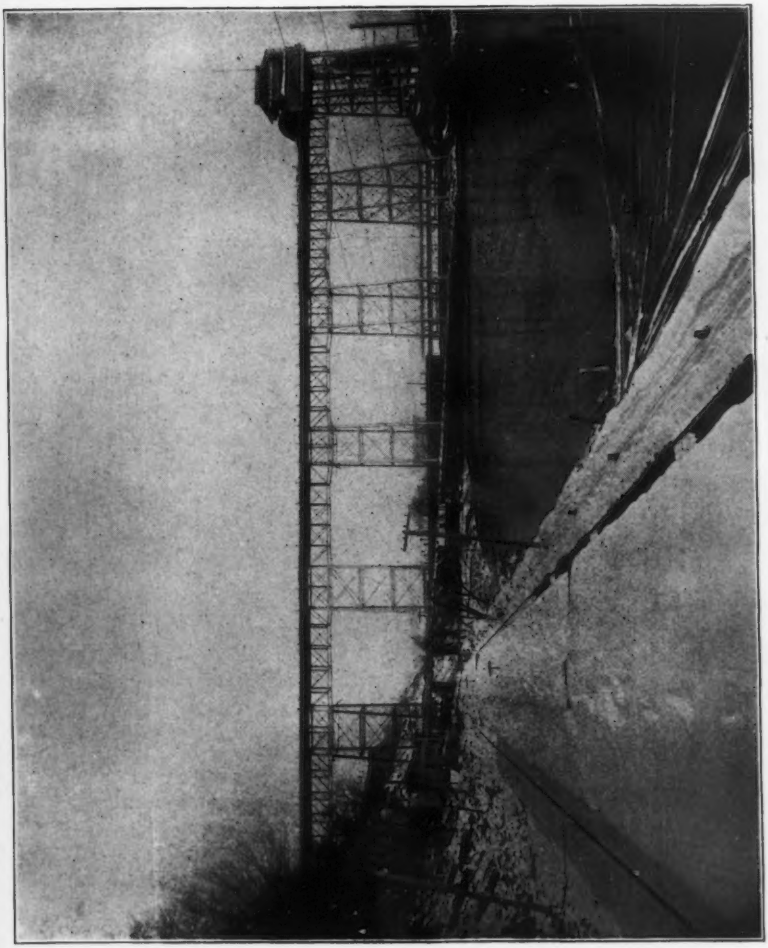


PLATE II.
TRANS. AM. SOC. CIV. ENGS.
VOL. XXVII, No. 535.
BLAKELEY ON WEEHAWKEN VIADUCT.



Northern Railroad of New Jersey, but at such a distance from the summit as to discourage the development of small residence towns. The amount of traffic which the construction of this viaduct, and the extension of a railway northward might reasonably be expected to develop in this locality, can be surmised from the fact that a similar extent of territory tributary to the inclined elevated railway at Newark Avenue, Hoboken, belonging to the same company, yielded during the past year a revenue of $400 000.

The desirability of the summit of the Palisades for residence is unquestionable, when its proximity to New York and its elevation are considered, but its inaccessibility has thus far prevented its development. The great amount of the suburban traffic to points in New Jersey, less favorably situated than this, will be apparent when it is considered that between the hours of 4 P.M. and 7 P.M. there is a train leaving the west side of the Hudson River every two minutes for suburban towns. It was in view of these considerations, and the possibility of increased growth of traffic in the future, that the structure was designed for the ordinary local traffic on surface steam railways.

It was at first proposed to use wrought iron throughout, but upon investigation it was found that steel would be more economical, and that a structure of mild steel could be built to provide for the heavier loads at a slight increase of cost over that of a wrought-iron structure designed for the lighter loads. In an appendix, a table is given of some tests made to show the relative effects of punching on iron and steel specimens, which were made by the writer, in relation to this subject, at the request of the President of the North Hudson County Railway Company.

The structure as it now stands, both elevator plant and viaduct, was constructed from the plans and designs prepared by Thomas E. Brown, Jr., M. Am. Soc. C. E.

Substructure.—The piers for the towers, with the exception of those for the elevator tower, and the tower next adjacent to it on the west, rest upon rock. The foundations for these two towers were secured by means of piles driven to bed rock. The west abutment consists of a pocket blasted into the igneous rock of the Palisades, and leveled up with concrete.

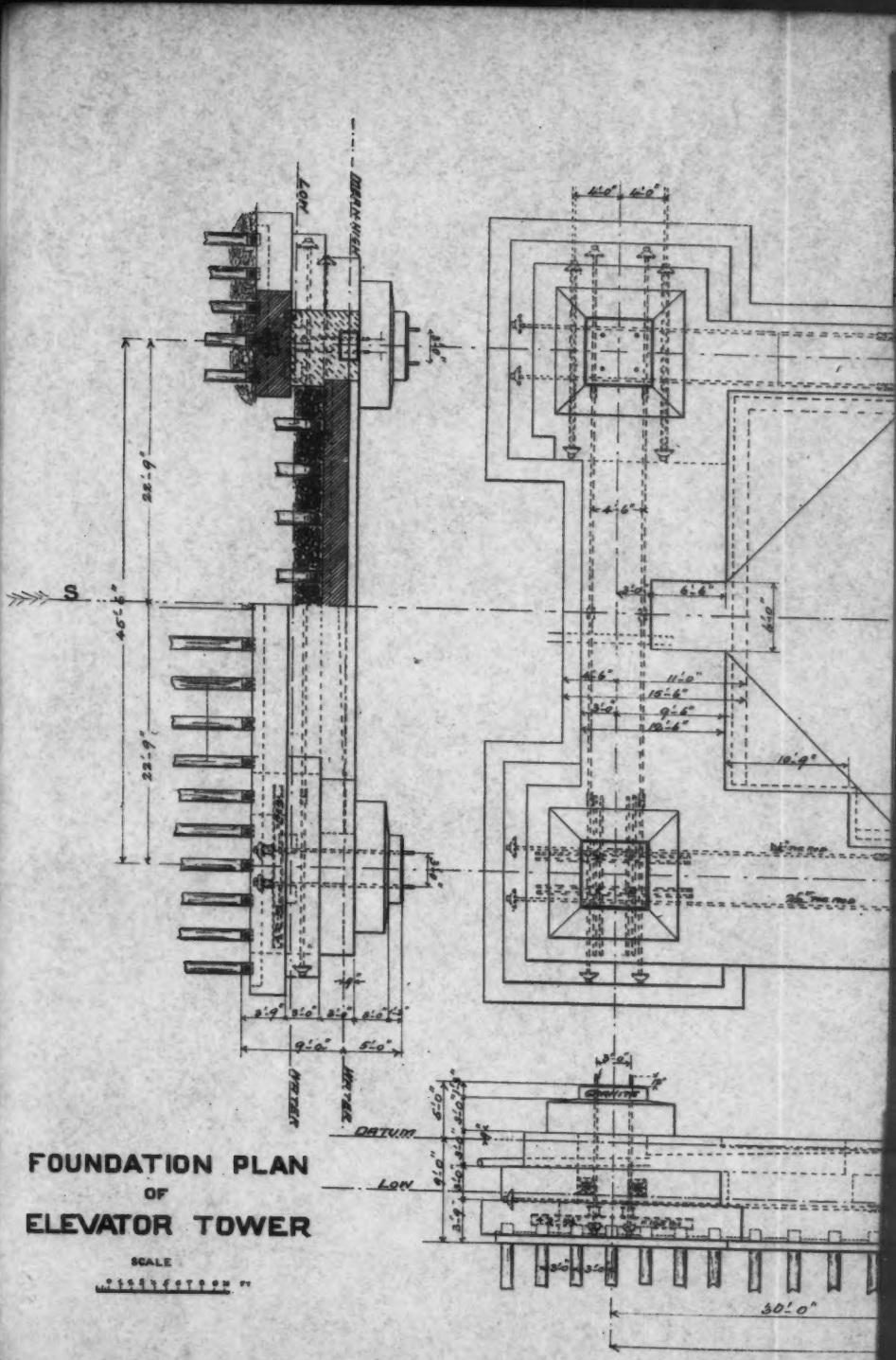
Each corner pier of the elevator tower rests upon forty-nine piles arranged in a square of seven rows of seven each, 3-foot centers, driven

to bed rock, which was reached at an average depth of 70 feet below the surface. The piles were sawed off 4 feet below low-water line and capped with two rows of 12 x 12-inch timbers laid at right angles. All timber in the foundations is kept below low-water line. Special foundations for pumps, elevator machinery and boiler house were provided at this tower, connecting with the corner foundations, and were secured in practically the same manner. The piers for the elevator tower are of concrete in four masses of full size, and of 3 feet thickness each, upon which rests a granite cap, 6 feet square and 15 inches thick. Each pier is tied in either direction to the other piers with two tie rods, 2¼ inches in diameter (Plate III).

The piers for tower No. 2 each rest upon thirty piles, arranged in five rows of six each, 3 feet and 2 feet 6 inches centers, capped in the same manner as those for the elevator tower. Bed-rock was reached under this tower at an average depth of 30 feet below the surface. The foundations are constructed of concrete arranged in three steps: the bottom mass is 15 feet square and 4 feet thick; the middle mass is 12 feet square and 3½ feet thick, and the top mass is 9 feet square and 3 feet thick. The pedestal stone, upon which the base of the tower column rests, is of granite, 6 feet square and 15 inches thick. Four anchor bolts, 2½ inches in diameter, are built into the concrete and secured to an anchorage imbedded in the lower mass of concrete. The grillage on top of the piles is also imbedded in this lower mass of concrete.

The above is the general type of pier, but modifications were made to suit the local conditions. Where rock was accessible, the lower mass of concrete was laid upon the dressed surface of the rock. In two of the foundations, gravel was the bottom secured, upon which was spread a bed of concrete 12 inches in thickness, and over which a solid floor of 12 x 12 timbers was laid. On top of this layer of timber the lower mass of concrete for the foundation was placed.

The foundations were constructed by the North Hudson County Railway Company under their own supervision. The concrete was mixed by hand in the proportion of two barrels of German Portland cement, three barrels of sand, and five barrels of broken stone. The quantities were measured by removable gauges in the box used for mixing. A cubic foot of the concrete taken from the mixture in use for the foundations, weighed 148 pounds, two weeks after setting.



FOUNDATION PLAN OF ELEVATOR TOWER

SCALE

PLATE III
TRANS. AM. SOC. CIV. ENGRS
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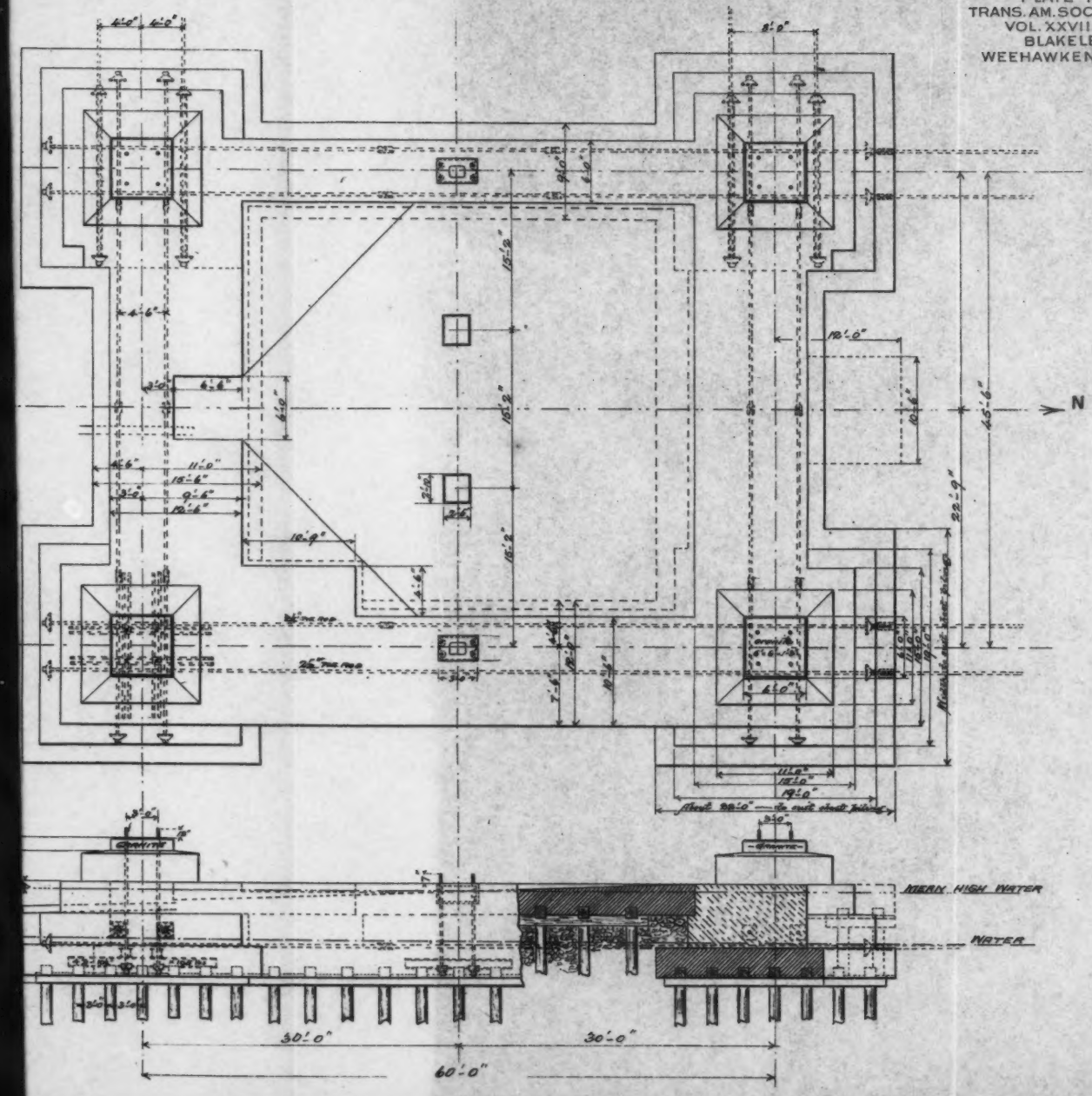
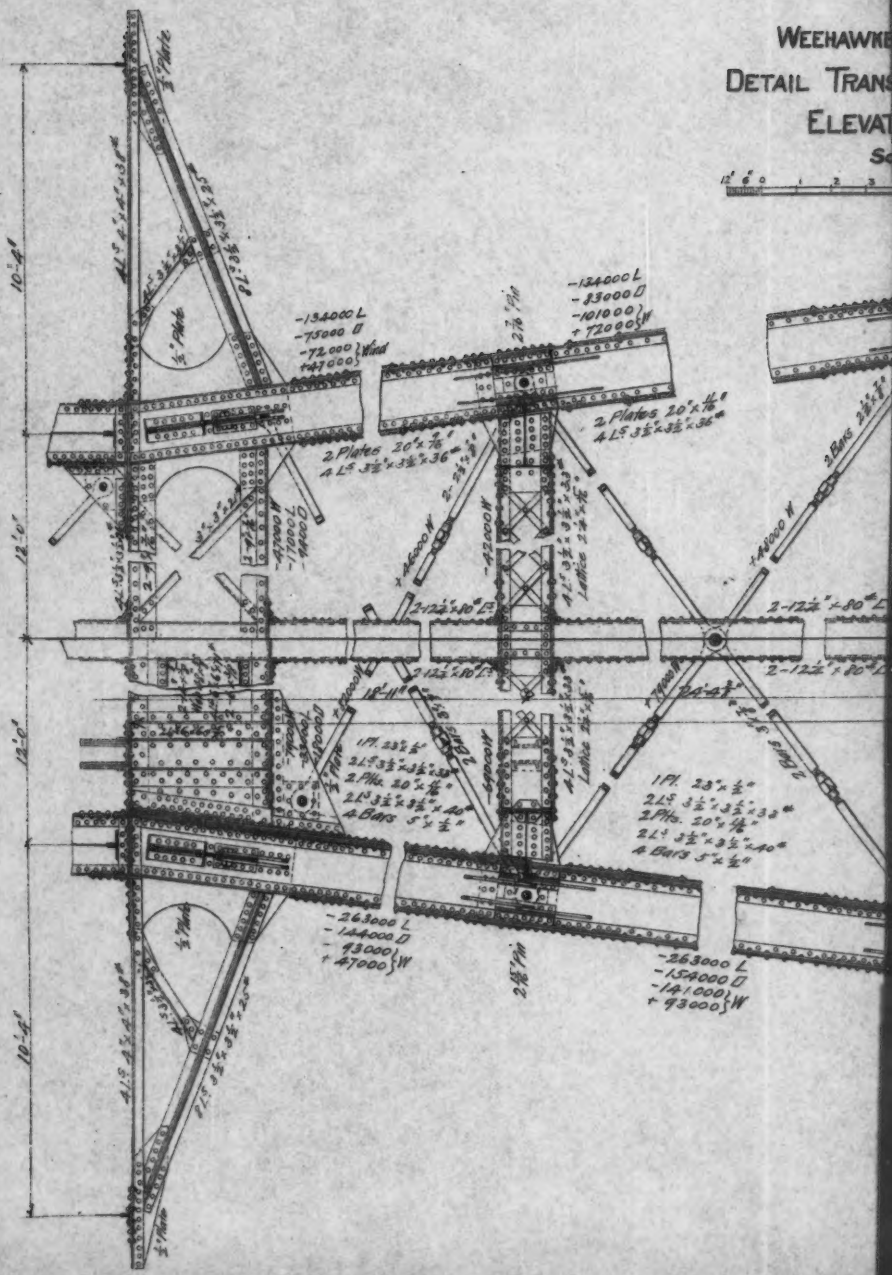
BLAKELEY ON
WEEHAWKEN VIADUCT.




WEEHAWKE
DETAIL TRANS
ELEVAT
SC
-134000 L
-75000 D
-72000 } Wind
+47000
-134000 L
-83000 D
-101000 } W
+72000
2 Plates 20"x 7/8"
4 L's 3½"x3½"x36"
2 Plates 20"x 7/8"
4 L's 3½"x3½"x36"
+46000 W
-42000 W
2-12½"x80" L
2-12½"x80" L
1 Fl. 23½"x½"
2 L's 3½"x3½"x38"
2 Plts. 20"x½"
2 L's 3½"x3½"x40"
4 Bars 5"x½"
1 Fl. 23"x½"
2 L's 3½"x3½"x38"
2 Plts. 20"x½"
2 L's 3½"x3½"x40"
4 Bars 5"x½"
-263000 L
-144000 D
-93000 } W
+47000
-263000 L
-154000 D
-141000 } W
+93000
10'-4"
12'-0"
12'-0"
10'-4"

PLATE
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VOL. XXV
BLAKEL
WEEHAWKE

WEEHAWKEN VIADUCT

DETAIL TRANSVERSE SECTIONS

ELEVATOR TOWER

SCALE

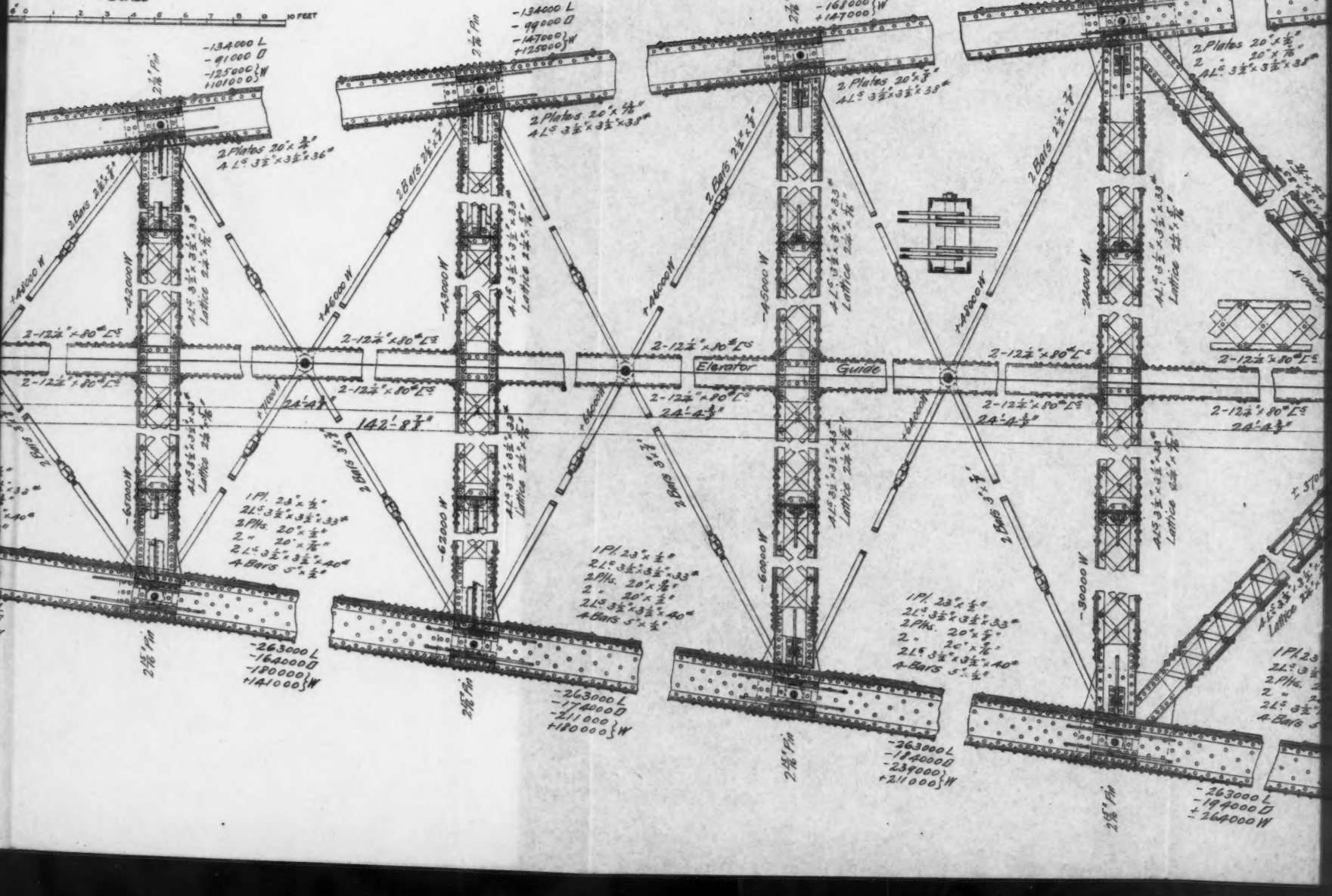

PLATE IV
TRANS. AM. SOC. CIV. ENGRS
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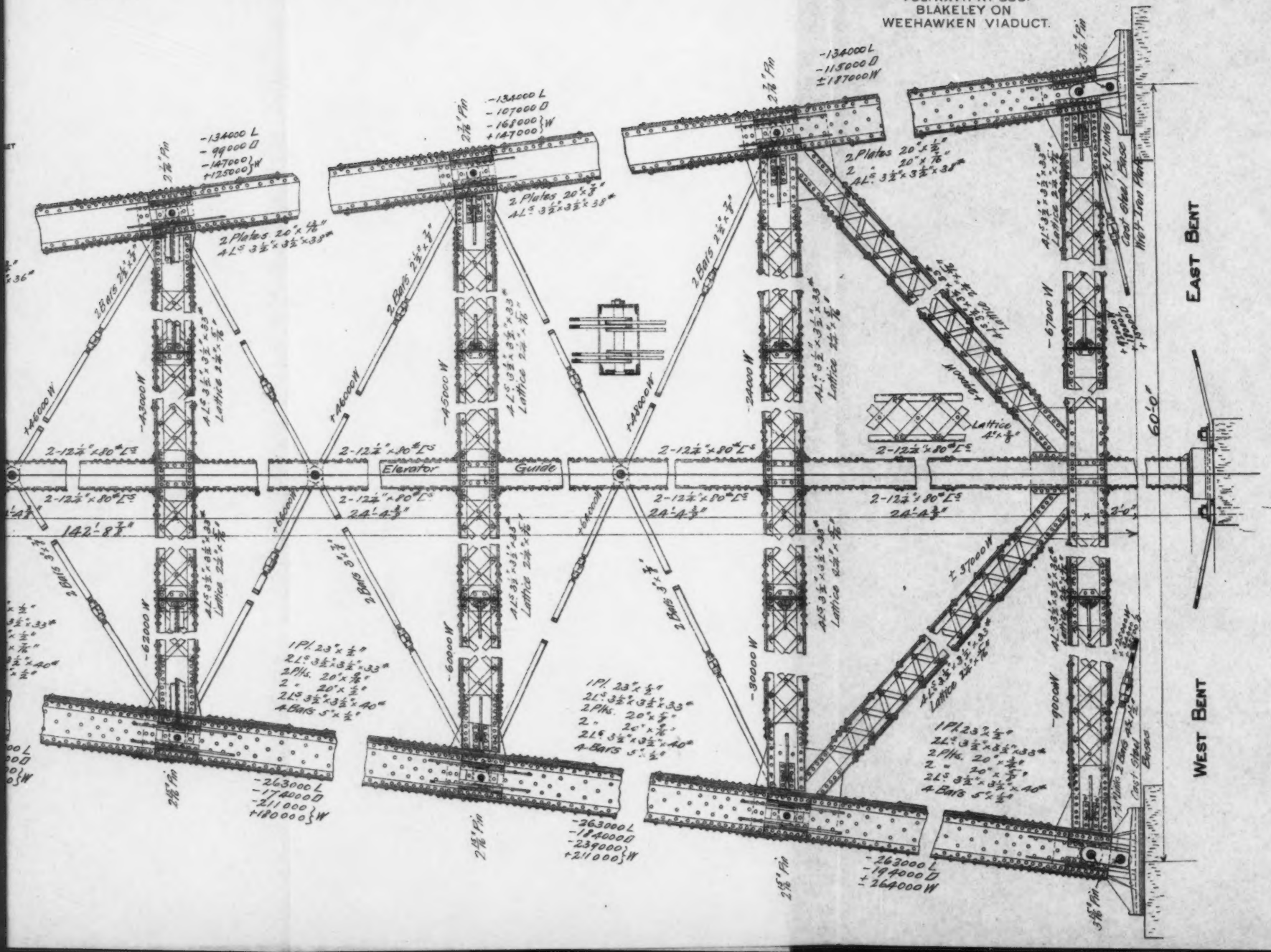
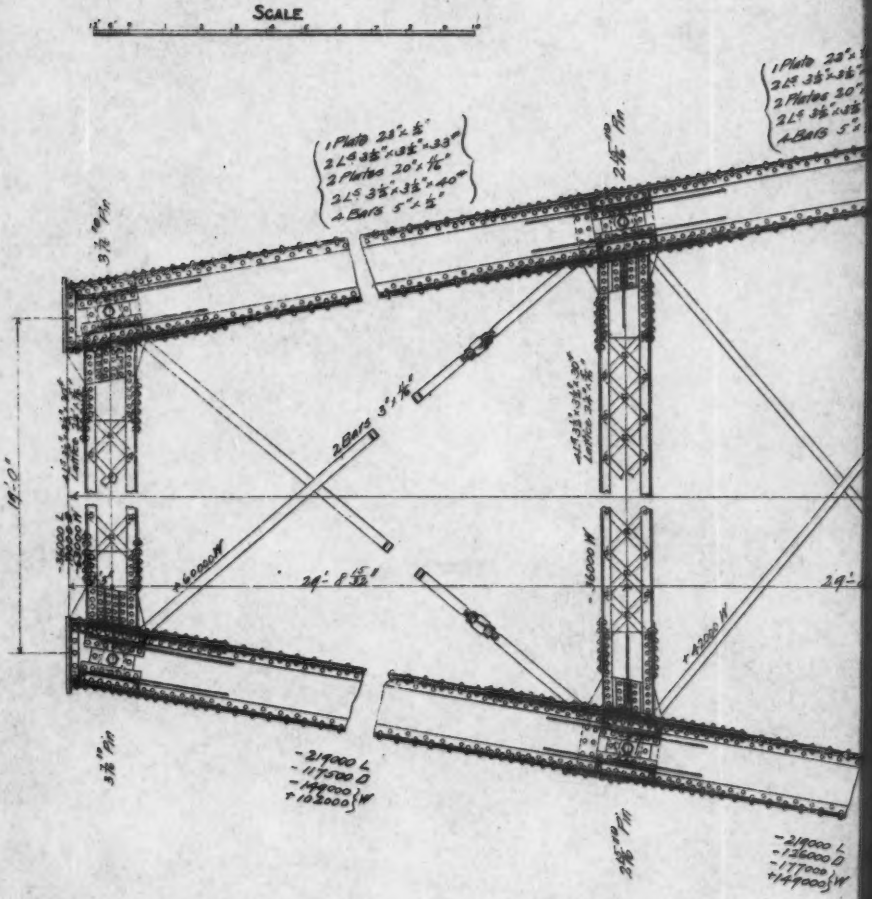
BLAKELEY ON
WEEHAWKEN VIADUCT.



DETAIL
VERTICAL SECTION OF REGULAR TOWER

SCALE



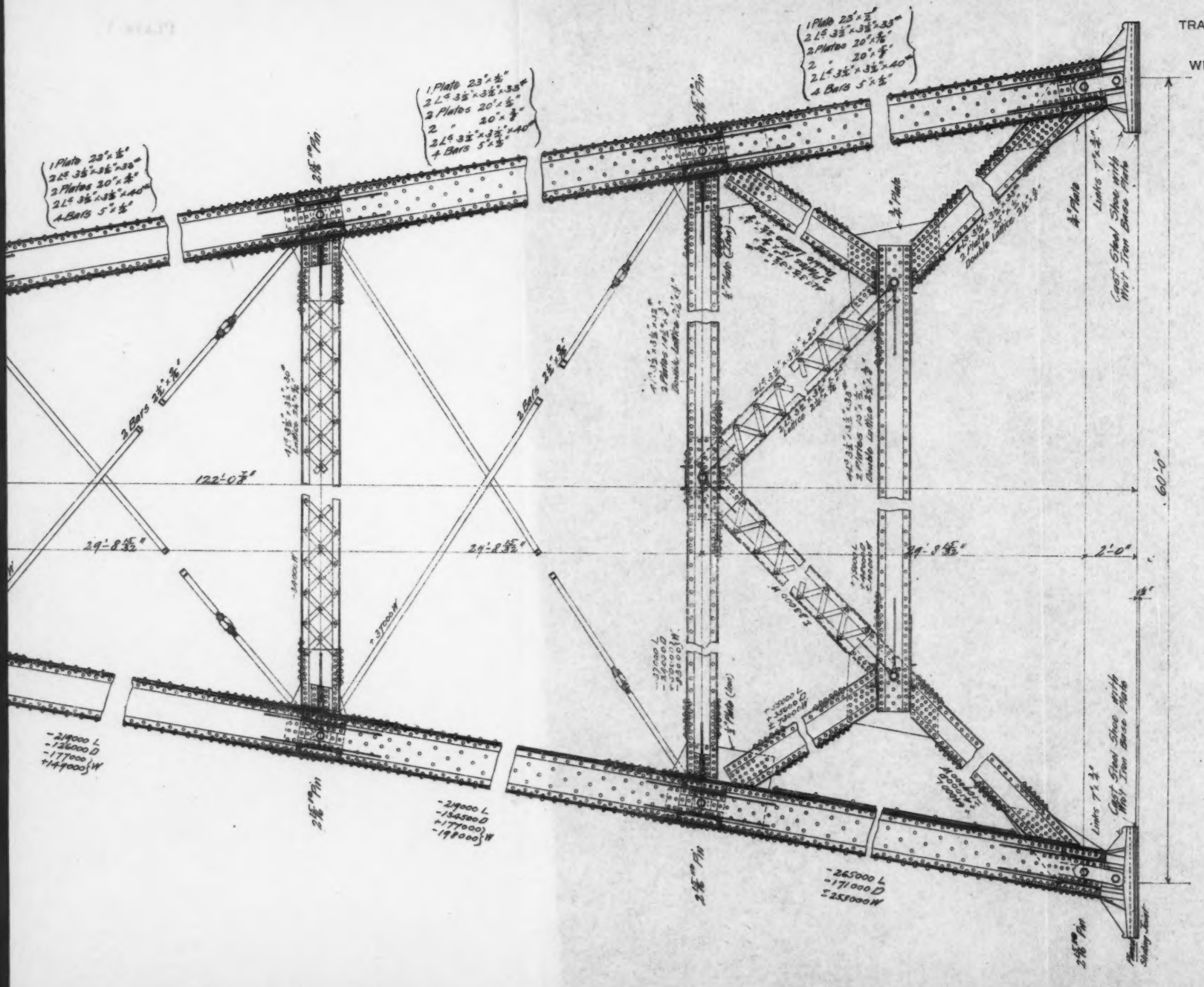


PLATE V
TRANS. AM. SOC. CIV. ENGRS
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BLAKELEY ON
WEEHAWKEN VIADUCT.
60'-0"
122'-0 7/8"
29'-8 15/32"
2'-0"
24" Pin

The maximum load upon each of the corner piers of the elevator tower is 628 tons, or an average load of $12\frac{8}{10}$ tons upon each pile. The maximum load upon each of the piers for the intermediate towers is 470 tons, or an average load of $15\frac{6}{10}$ tons upon each pile. The above loads are based upon the maximum combined effects of live and dead loads, and wind, together with the weight of the pier, which is taken at 148 pounds per cubic foot.

Superstructure.—The structure is composed of one tower containing the elevators, of 45 feet 6 inches length; four towers of 40 feet; one tower of 35 feet; two spans of 80 feet; two spans of 100 feet; one span of 93 feet, and one span of 180 feet. The total length of the structure from the center of the end pin on the west abutment to the center of the eastern bent of the elevator tower is 873 feet 6 inches. The height of the top of the pedestal stones above the datum (mean high water) is 5 feet. The base of rail at the west abutment is at an elevation of 150 feet, from which point there is an upward grade of 3 feet in the length of the structure, making the height of the base of the rail above mean high water at the eastern end, 153 feet.

The elevator tower is 60 feet in width at the base, and 24 feet at the height of the railway (Plate IV). The batter is $1\frac{1}{2}$ inches to the foot. The extreme height from the top of the foundation to the ridge of the station upon the top of the tower is 194 feet 2 inches. This tower was designed to carry the loads, and to suit the conditions imposed upon it by the elevators and elevator machinery. The viaduct towers are 19 feet in width at the top, and 60 feet at the base (Plate V). The batter is 2 inches to the foot. For convenience of construction, the dimensions of these towers and the depths of the spans between them were made uniform throughout; and, to provide for the grade in the structure, the towers were built for the lowest point, and the difference in elevation at the top was made in the depth of the spans over the towers and in the inclination of the end bottom struts of the main spans.

The location of the towers had been decided upon and the legal arrangements for right of way perfected, before any consideration was given to the question of the economic lengths of spans. The structure is built to conform to the locations agreed upon prior to construction. For economy of material, the lengths of the spans should have been 125 feet.

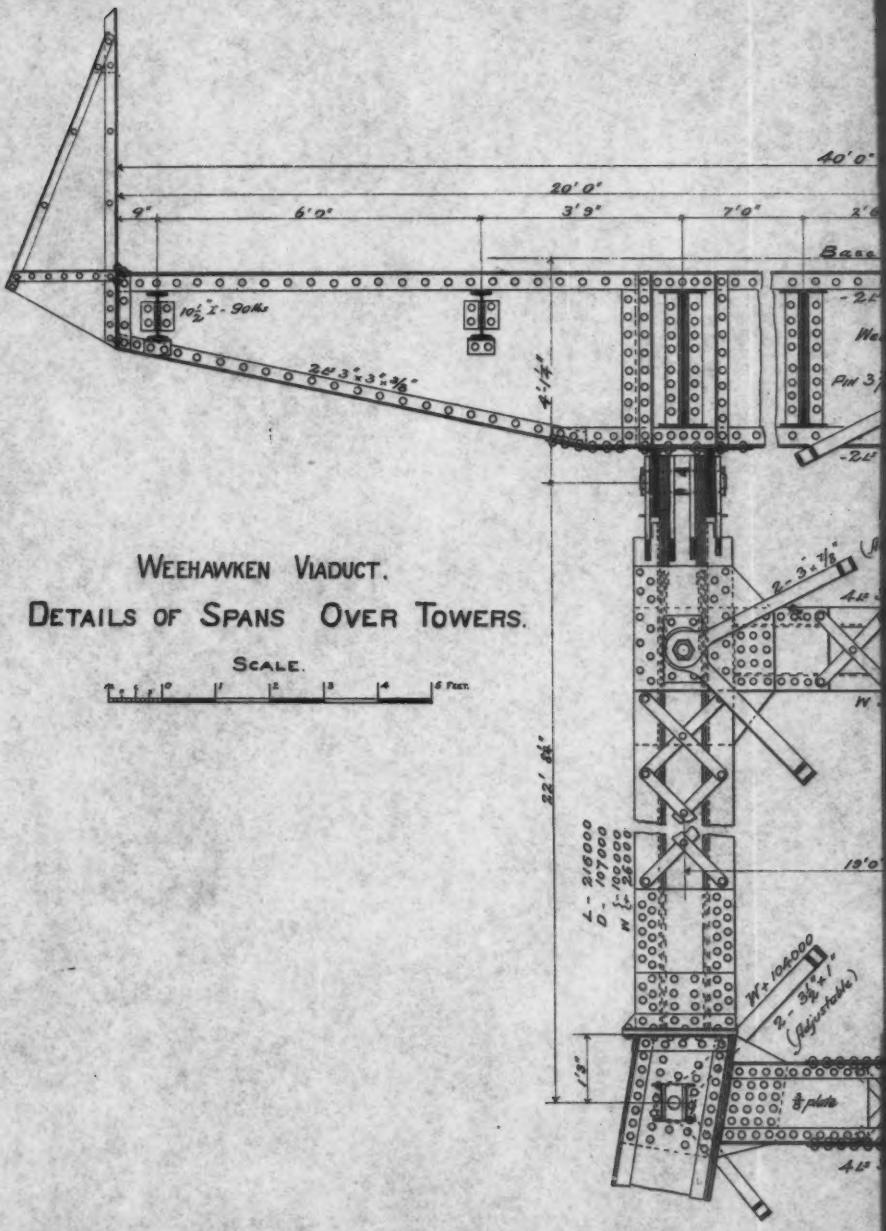
The concessions for right of way made the provision for a roadway under the structure obligatory. The portals in the lower part of the towers were designed to meet this requirement, and, at the same time, provide for the transverse bracing of the tower, independently of the bending resistance of the main columns. The northerly face of the tower over the West Shore round house spans an engine stall, and permits the passage of a locomotive. A similar portal was used as in the transverse bracing, only it was not possible to bring it down to the foot of the tower columns, which are re-enforced on the webs to resist the bending thus induced.

The foot of each leg of the tower abuts upon a steel pedestal casting, which rests upon a wrought-iron base plate $2\frac{1}{4}$ inches in thickness. Both the top of the casting and the foot of the column are faced, and the latter is held in place horizontally by a tenon on the casting, which fits closely around the inside of the column. The bottom of the steel-casting is planed, as is also the top of the wrought-iron plate. One corner of each tower is anchored securely against movement, while the other corners are free to move—both transversely and longitudinally, upon the sliding surfaces between the cast-steel pedestal and the wrought-iron base, but are secured against lifting, the provision being made by means of enlarged holes for the anchor bolts. The minimum pressure on these sliding surfaces is 80 pounds per square inch, and the maximum pressure is 300 pounds per square inch.

The towers are secured to the cast-steel bases with wrought-iron links shrunk on pins, passing through the column and the base, and of sufficient section to resist the maximum lifting force produced by the combined effects of wind and traction. Tension at the base of the towers exists at one corner only under these combined effects, and the anchorage provided is sufficient to resist it.

All the struts of the towers, both transverse and longitudinal, are composed of four angle irons latticed on four sides. The transverse struts are riveted to wing plates projecting from the columns. The longitudinal struts are connected to the posts with pins passing through a deep U-nut riveted to the posts, and are also riveted to these U-nuts, the rivets being driven after the erection of the structure.

The trusses of the spans are spaced 19 feet on centers, and are 20 feet deep center to center of chords. The stringers under each track

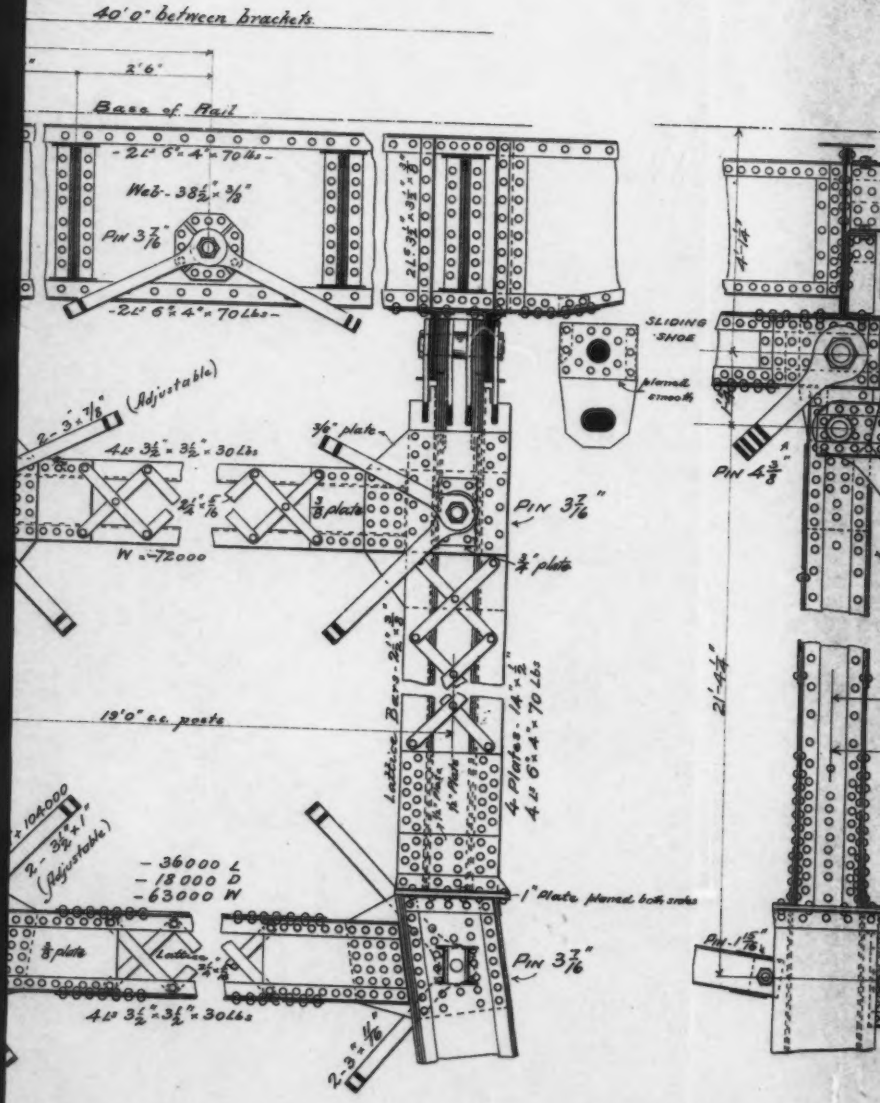


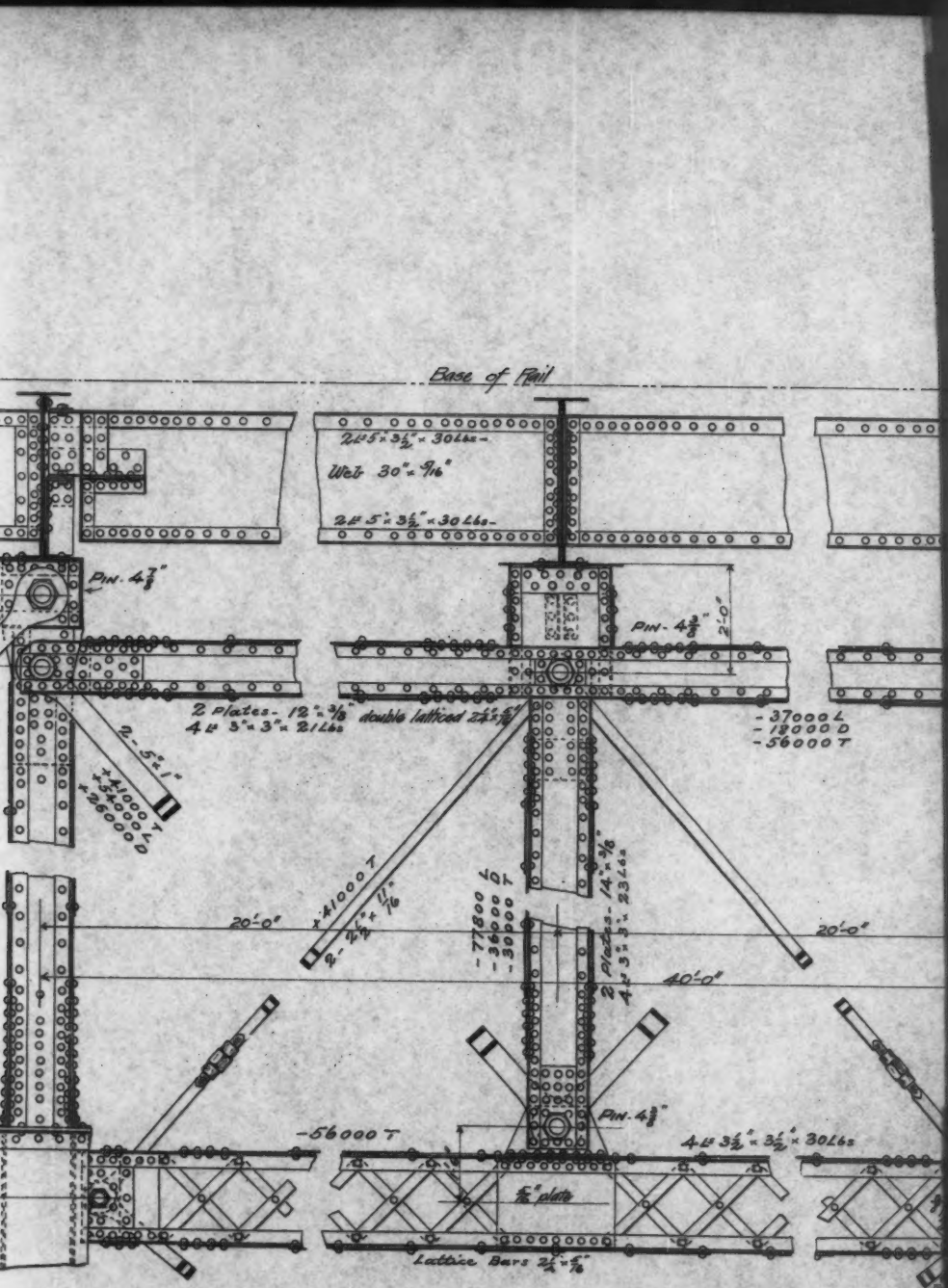
WEEHAWKEN VIADUCT.

DETAILS OF SPANS OVER TOWERS.

SCALE.

0 1 2 3 4 6 FEET.


40'0" between brackets
2'6"
Base of Rail
-2L⁵ 6"x4"x 70 Lbs-
Web - 38½" x ⅜"
Pin 3 7/16"
-2L⁵ 6"x4"x 70 Lbs-
2L⁵ 3½"x3½"x ⅜"
SLIDING SHOE
Planed smooth
4'-4½"
2-3"x⅞" (Adjustable)
4L⁵ 3½"x3½"x 30 Lbs
3/4" plate
Pin 4⅜"
2¼"x⅝"
⅜ plate
Pin 3 7/16"
W = 72000
¾" plate
19'0" c.c. posts
Lattice Bars - 2½"x⅜"
4 Plates - 14"x⅝"
4L⁵ 6"x4"x 70 Lbs
21'-4½"
= 104000
2-3½"x1" (Adjustable)
- 36000 L
- 18000 D
- 63000 W
1" Plate planed both sides
⅜ plate
Lattice
Pin 3 7/16"
Pin 1 15/16"
4L⁵ 3½"x3½"x 30 Lbs
2-3"x 1/16"


Base of Rail
2 ⌐ 5" × 3½" × 30 Lbs.
Web 30" × 9/16"
2 ⌐ 5" × 3½" × 30 Lbs.
Pin 4 7/8"
Pin 4 3/8"
2'-0"
2 Plates - 12" × 3/8"
4 ⌐ 3" × 3" × 21 Lbs
double latticed 2½" × 6"
- 370000 L
- 130000 D
- 560000 T
2 - 5" × 1"
+ 410000 T
+ 300000 D
+ 260000 D
+ 410000 T
4' - 4½" + 1½"/16
20'-0"
- 77000 L
- 360000 T
- 300000 D
2 Plates - 14" × 3/8"
4 ⌐ 3" × 3" × 23 Lbs
20'-0"
40'-0"
Pin 4 1/8"
- 560000 T
4 ⌐ 3½" × 3½" × 30 Lbs
5/8" plate
Lattice Bars 2½" × 7/16

TOWER Nº 2.

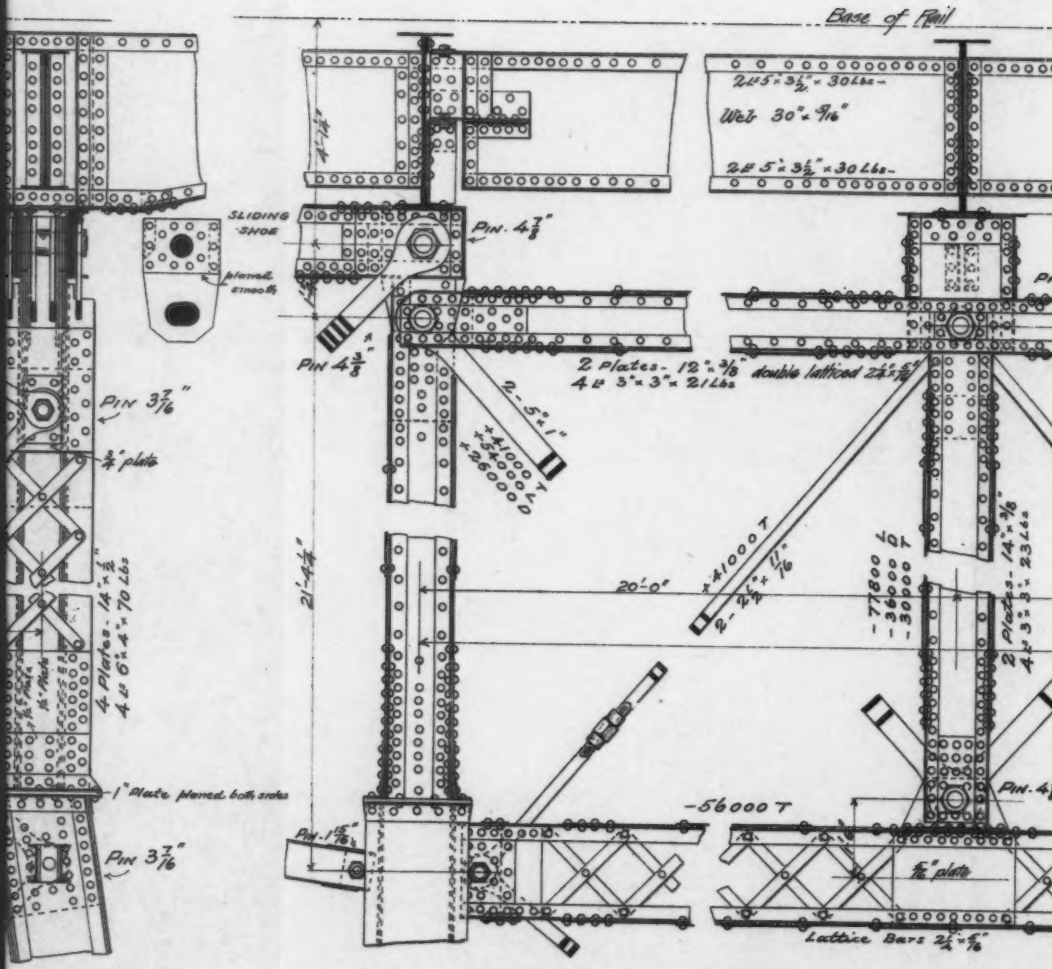

Base of Rail
SLIDING SHOE
planed smooth
Pin 4 7/8"
Pin 4 3/8"
Pin 3 7/16"
3/4" plate
4 Plates - 14" x 3/8"
4 L 6" x 4" x 70 Lbs
1" Plate planed both ends
Pin 3 7/16"
2 L 5" x 3 1/2" x 30 Lbs
Web 30" x 9/16"
2 L 5" x 3 1/2" x 30 Lbs
2 Plates - 12" x 3/8"
4 L 3" x 3" x 21 Lbs
double latticed
2 - 5" x 1"
+410000 T
+260000 T
21'-4 1/4"
20'-0"
+410000 T
4' 4 1/4" x 11/16"
-77800 L
-36000 T
-30000 T
2 Plates - 14" x 3/8"
4 L 3" x 3" x 23 Lbs
-560000 T
Pin 1 15/16"
5/8" plate
Lattice Bars 2 1/4" x 5/16"
TOWER

PLATE VI
TRANS. AM. SOC. CIV. ENGRS
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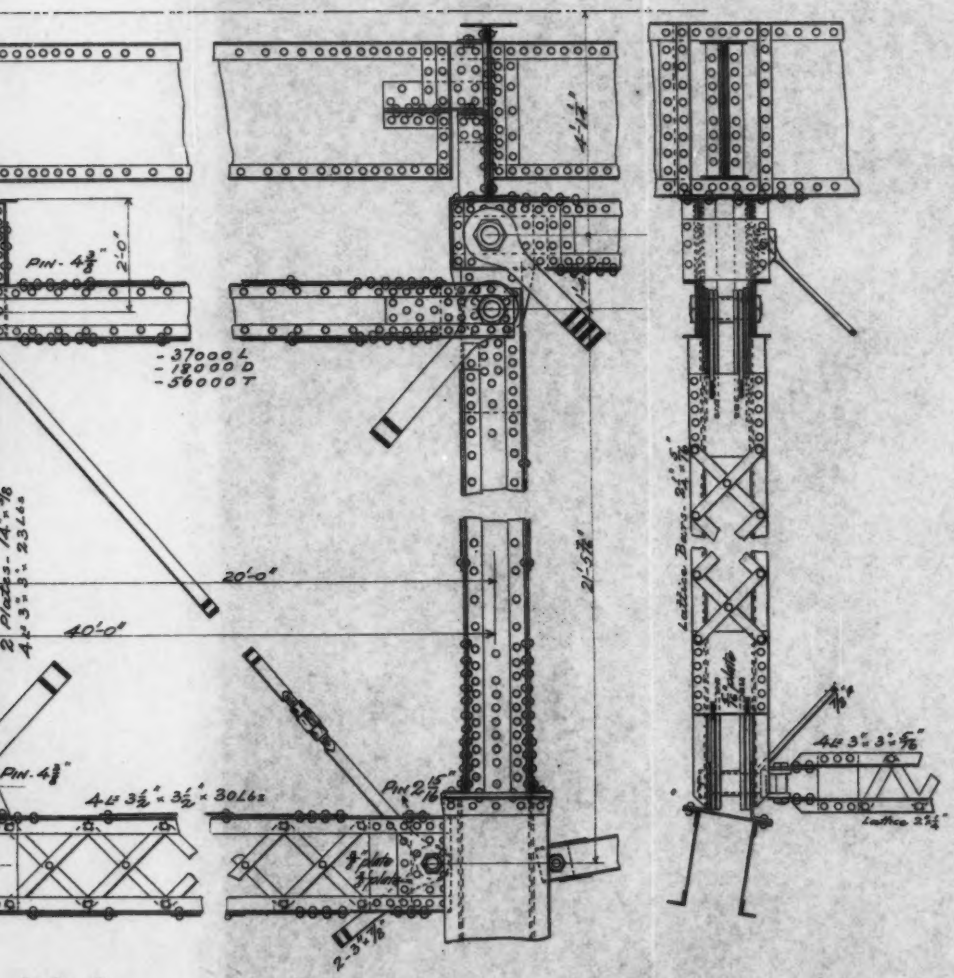
BLAKELEY ON
WEEHAWKEN VIADUCT.

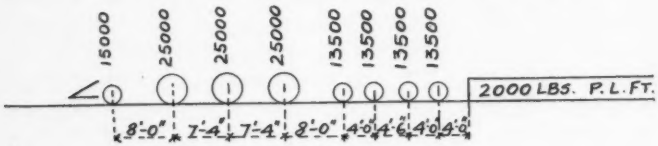


Nº 2.

are spaced 7 feet centers. Cross-floor beams rest upon the top chords of the trusses with projecting brackets for carrying sidewalks 6 feet in width, on each side of the tracks. The expansion of the trusses at the towers is by means of a shoe sliding upon the planed top of the vertical post of the towers. This shoe is free to move sufficiently to provide for the expansion and contraction produced by 150 degrees change of temperature, but is secured from movement beyond that point. The maximum pressure on the surface of the sliding joint is 6 600 pounds per square inch; the minimum pressure is 2 200 pounds per square inch (Plate VI).

Pin-connected trusses were used for spanning the towers, in order to give uniformity in appearance, and, at the same time, provide a satisfactory method for the transfer of the strains produced by traction. The 180-foot span was rendered necessary by the uncertainty of securing proper abutments on the face of the Palisades. The rock at first gave evidence of being solid, but at the beginning of blasting operations, it scaled back and revealed a serious crack which separated it from the body of the hill. This uncertain rock was entirely removed, and foundations secured upon certain and solid rock. The depth of the 180-foot span (viz., 20-foot centers of chords), while manifestly very shallow for its length, was adopted to avoid what was considered by the officers of the railroad company the bad appearance produced by changing the depth, which was kept uniform in the rest of the structure.

The structure was designed to carry, in addition to its own weight, a double-track standard gauge railroad, each track being loaded with a train consisting of a 72-ton Mogul locomotive, followed by a train load of 2 000 pounds per lineal foot, as per the following diagram:



And, in addition to the above, a moving live load of 80 pounds per square foot of sidewalks was provided for. The sizes of the members were in all cases proportioned according to Mr. Theodore Cooper's "Specifications for Railroad Bridges and Viaducts." The columns of the towers were proportioned to resist the strains produced by live

load, dead load, and wind, the strain from each of these loads being divided by the proper factor given in Mr. Cooper's specification, and the separate results added to form the aggregate section required for the column. The strains produced by traction were not considered in proportioning the columns, as, by the application of Mr. Cooper's formula for the strains allowable on "posts," the sections were considered already more than amply large.

Open-hearth acid steel was used throughout except for adjustable rods and bars, lattice bars, batten plates and rivets, which were of wrought iron. In employing steel it was not required that punched holes be reamed, or that sheared edges be planed. The following is extracted from the proposal for furnishing the material submitted by the manufacturers:

"The steel will be produced by the open-hearth acid process, and no specific requirements of its chemical constitution shall be required, provided it fulfills the following physical tests, viz.:

"When tested in standard test pieces having an area of one-half square inch or more, it shall have an ultimate strength of not less than 62 000 pounds per square inch, and not more than 70 000 pounds; an elastic limit of not less than 30 000 pounds per square inch; a minimum elongation of 20 per cent. in an original length of 8 inches, and a minimum reduction of area of 40 per cent.

"It shall stand satisfactory bending and quenching tests as prescribed by Cooper's specifications.

"The planing of sheared edges and the reaming of punched rivet holes shall not be required, provided the material permits the drifting of a punched hole, as ordinarily placed, to an increased diameter at least 25 per cent. greater than the original diameter without cracking."

The cost per ton of the steel structure under these conditions was the same as for iron.

The actual weights of the metallic superstructure, exclusive of the elevator machinery are as follows:

	Pounds.
Elevator Tower	963 046
Tower No. 2	362 159
Tower No. 3	317 639
Tower No. 4	296 963
Tower No. 5	306 539
Tower No. 6	335 097
Two spans of 80 feet	283 761
Two spans of 100 feet	324 757
One span of 93 feet	152 952
One span of 180 feet	441 133
Total	3 784 046

The weight of the span over the tower is included with each tower.

Erection.—As it was desirable that the tower containing the elevators be the first erected—to enable the elevator machinery to be placed in position as soon as possible, in order to be ready for operation at the completion of the viaduct—the erection of the structure was begun on the east end and proceeded westward toward the bluff. False work was used in the erection of the spans; two entire openings between towers were filled with false work at the same time, and also a part of the third opening; this left two spaces for towers. Into these spaces booms projected, which were lashed fast to the false work from which the tower posts and transverse bracing were lifted into place, and the vertical bent, thus formed, lashed fast to the false work on the side adjacent, until the longitudinal bracing was lifted into place by the traveler on the top of the false work.

On the top of the false work were two travelers; one of the ordinary type used in the erection of through span bridges with the addition of two booms swinging at the front for the handling and hoisting of material from the side, and the other a traveler of the double **A** type, with booms for the same purpose swinging at the rear. The width of the towers on top being but 22 feet over all, while some of the floor beams were 50 feet in width, it was unadvisable to construct the false work for a traveler sufficiently wide to pass these floor beams. The first traveler erected the trusses and lateral bracing, while the second traveler, following behind it, swung the floor beams and stringers into place. Material was hoisted at the sides from a track on the ground running parallel with the structure. A boom derrick was put in place at the track level, near the elevator tower, for hoisting the material required in the upper part of that tower, as also the track material, ties, rails, etc. Much of the iron-work was hoisted by this derrick and run out over the completed structure to the travelers. The travelers were carried across the towers by means of a temporary wooden truss, as at these points the run-way for the traveler was wider than the top of the tower.

The false work was of yellow pine, each bent having two vertical legs, 10 x 10 inches, and two batter legs of the same dimensions. The bents of false work were placed 18 feet 5 inches apart on centers, and tied together longitudinally with bracing of 3 x 10-inch timbers in alternate bays. It was framed accurately according to drawing, in order to

facilitate its handling and erection. As soon as a span was completed and the trusses swung, the false work was taken down and put up for one of the spans in advance.

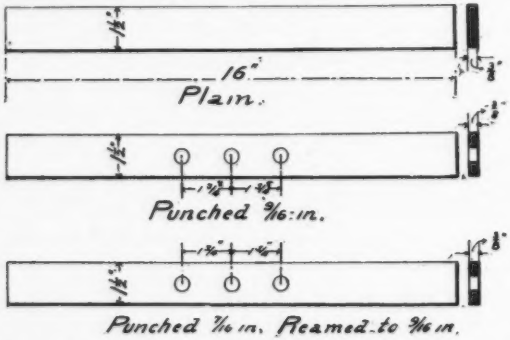
The hoisting of all the material in erection, including the false work, was done with steam, from engines located at different parts of the work.

The metallic superstructure was manufactured and erected by the Passaic Rolling Mill Company of Paterson, N. J., under the supervision of the writer.

APPENDIX.

Each set of specimens, the average of the tests of which are given in the table, consisted of six pieces, $2\frac{1}{2}$ inches in width, cut from the same plate and as close together as possible. After the specimens were planed to $1\frac{1}{2}$ inches in width, two from each set were punched, with holes $\frac{9}{16}$-inch diameter, and two were punched $\frac{7}{16}$-inch, and reamed to $\frac{9}{16}$-inch diameter.

The sizes of the test specimens were as shown:



The six specimens forming a set were tested, two plain, two punched, and two reamed. Eight sets of such specimens were prepared and tested, four of them being cut from iron plates, and four from steel plates.*

The table gives the average results for elastic limit and ultimate strength for each set, and the last four columns show the change in per cent. of these properties for the punched and reamed specimens, as compared with the plain specimens.

* A complete record of these tests was published in the *Railroad Gazette*, 1890.

AVERAGE RESULTS.

Material.	No. of set.	Limit of elasticity, pounds, per square inch. [Average of each set.]			Ultimate strength, pounds, per square inch. [Average of each set.]			Per cent. of change in limit of elasticity.		Per cent. of change in ultimate strength.	
		Plain.	Punched.	Reamed.	Plain.	Punched.	Reamed.	Punched.	Reamed.	Punched.	Reamed.
Iron.	1	28 650	32 250	....	49 650	40 000	....	+ 12.6	....	— 19.4	....
"	5	26 150	27 300	25 850	49 400	40 100	46 700	+ 4.4	— 1.5	— 18.8	— 5.5
"	6	24 500	25 050	23 850	50 350	42 100	47 200	+ 2.2	— 2.7	— 16.4	— 6.3
"	7	26 200	27 350	25 850	49 000	42 800	45 100	+ 4.4	— 1.3	— 12.6	— 8.0
					Average of experiments on iron.			+ 5.9	— 1.8	— 16.8	— 6.6
Steel	2	30 550	32 350	32 100	73 550	72 400	74 600	+ 5.9	+ 5.1	— 1.6	+ 1 4
"	3	29 950	30 850	29 200	66 150	62 250	67 150	+ 3.0	+ 2.5	— 5.9	+ 1.5
"	4	31 850	35 200	35 750	73 100	70 300	76 250	+ 11.0	+ 12.2	— 3.8	+ 4.3
"	8	31 400	33 250	31 870	66 850	62 500	67 000	+ 5.9	+ 1.4	— 6.5	+ 0.2
					Average of experiments on steel.			+ 6.5	+ 4.1	— 4.5	+ 1.9

II.—THE HYDRAULIC PASSENGER ELEVATORS.

By Thomas E. Brown, Jr., M. Am. Soc. C. E.

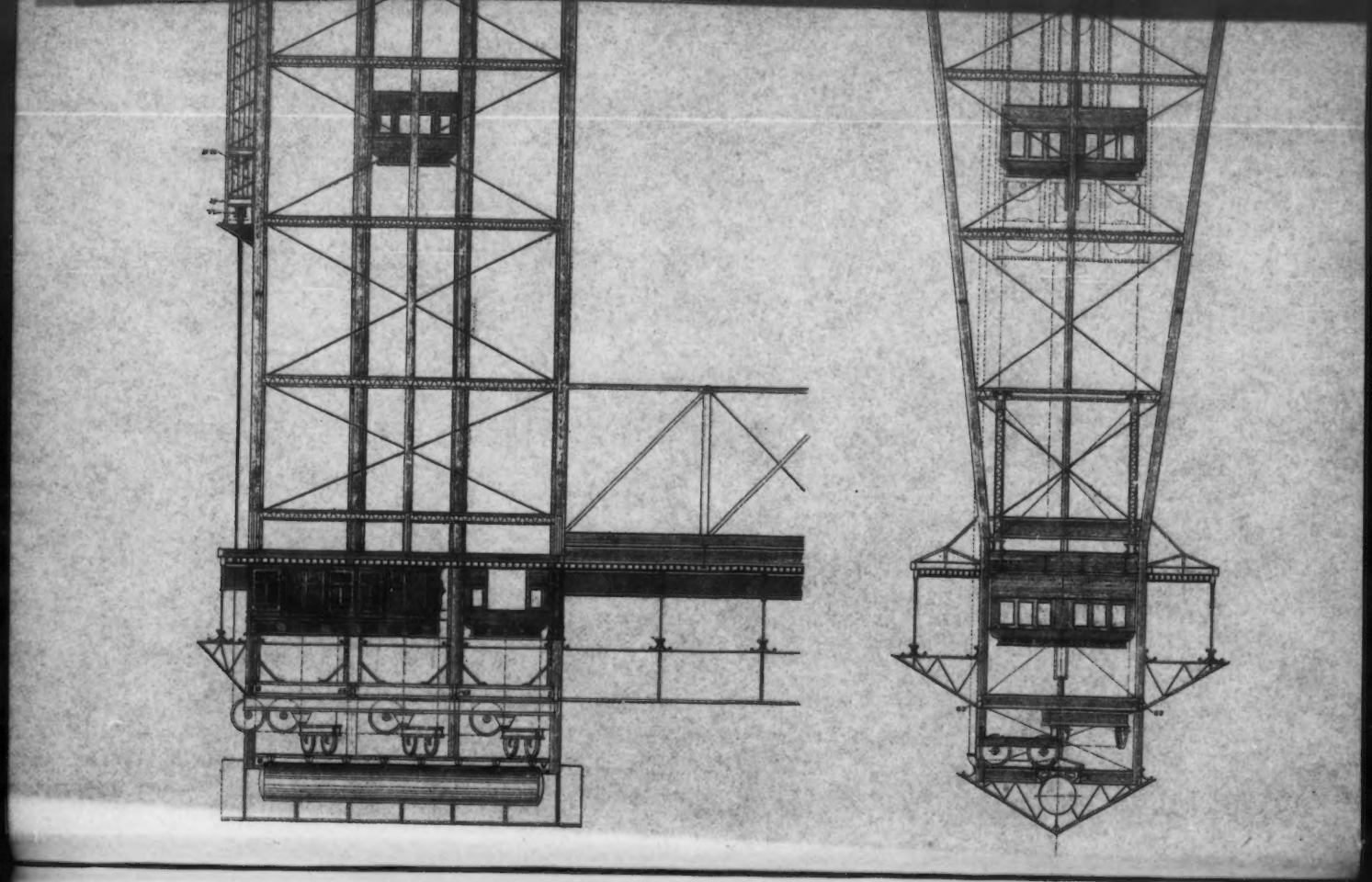
There are three passengers elevators, operated by water-pressure on the combined gravity and pressure-tank system from a compression-tank placed in the top of the tower. The rise from the ground floor level to the level of the upper station platform is 145 feet (Plate VII).

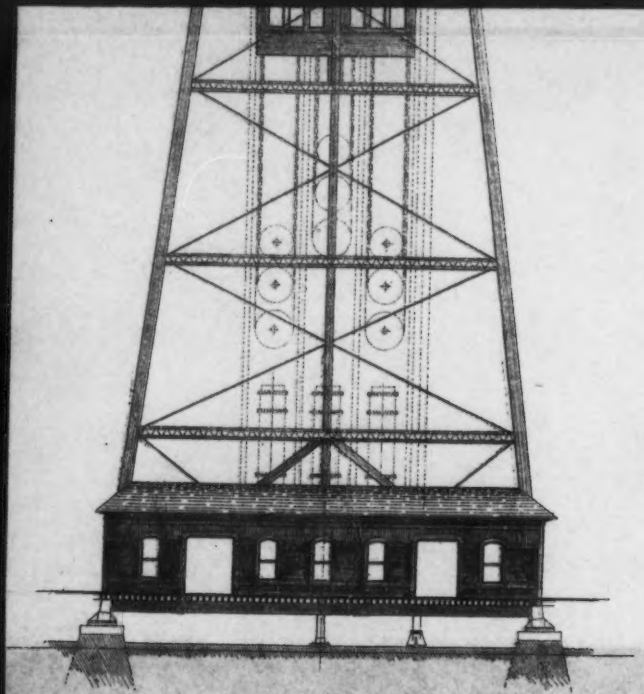
Each car is 21 feet 6 inches long by 12 feet 6 inches wide, and about 10 feet high, and is suspended in a frame consisting of four 8-inch channels, 50 pounds per yard, and lattice top and bottom cross-heads of 4 x 3-inch angle irons. This frame contains the rope attachments and safety-devices, and is of such proportions that, under ordinary service, no portion is strained to more than 9 000 pounds per square inch; and under the most extreme safety-strains no portion will be strained to exceed 18 000 pounds per square inch, and will support without exceeding this strain the entire weight of the car and load if held by the safety-devices on one side only.

The water-cylinders are of cast iron, 38 inches in diameter (Plate VIII). The joints are packed with flat rubber rings (see diagram, page 13). Each cylinder is secured to the foundation girders by two 4-inch anchor-bolts. Each head-plate is a heavy casting, strengthened by cross ribs, through which the piston rods work, in brass-lined stuffing-boxes. The head-plate is provided with apertures closed by bronze screw plugs, through which the piston-follower bolts may be screwed up, thus avoiding the necessity of removing the head-plate when screwing up the piston packing. The cylinders and their appurtenances were subjected to a hydrostatic test of 500 pounds to the square inch before leaving the shop.

The pistons are of cast iron, cast solid to act as counterweights, and are provided with automatic stop-aprons, which, by gradually closing the cylinder-ports, bring the pistons (and consequently the cars) gradually to rest at each end of the travel. This device is a late improvement, and has the advantage that it is entirely within the cylinder and directly attached to the initial moving part, viz., the piston, cannot be tampered with or lose its adjustment, and is independent of all other controlling devices.

The pistons are packed with braided flax packing. The piston is attached to the traveling-sheave frame by two turned steel piston-rods, 4¼ inches in diameter. The upper ends of the rods are forged





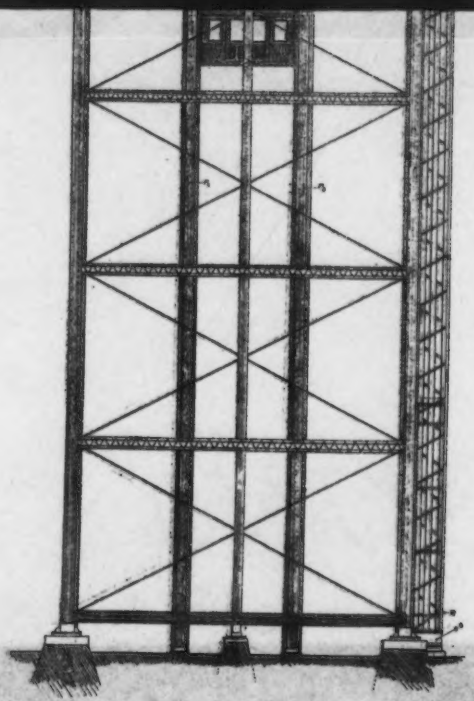


PLATE XIV

DIAGRAM OF TOWER

LOADS TO BE PROVIDED FOR

GIRDERS B.B. SUSPENSION TRUSS, BRACING OF THE TOWER POSTS P.R.R.P.	DISTRIBUTED LOAD 8000 lbs
CAPSTAN BALANCE SHEAVE AXLES	TRUE @ SHEAVE PINS

Diagram of Tower

Loads to be Provided for

Girders A.B. supporting Sand, resting on the four Posts P.P.P.P. Distributed Load	89000 lbs
Counter Balance Sheave Axles Vertical Load on each	7800 " Sailing Boats
" " " " Horizontal Thrust " "	7800 " 19 Gardens Creek
Main Sheave Axles Vertical Load on each	23000 " Hulks to be Accounted
" " " Horizontal Thrust " "	24000 " for these Loads
The resultant of Axle Loads see "A Double Geometric Plan"	
Main Posts M.M.M.M. Strain on each Post	51000 "
Posts P.P.P.P. in ordinary operation Vertical Load from Tarrit other	
" R.P.	14666 "
" P.R.	29333 "
Girders M.G.M. M.K.M. & G.H. IK. sustain No Loads in ordinary operation	
" L.S.M.M. sustain Distributed Load per Panel "	50000 "
Posts RR.,RR. " Direct Vertical Load each of	125000 "
" RR.,RR. " " " " "	63000 "
Girders O.O. supporting Cylinder Base, support Alternative and Overload Full "	93000 "
and a Vertical Downward Weight of	62000 "

Under Action of Safety Devices

Posts P.P.P.P. sustain assumed Tenth Weight Rivets above —	
A Distributed Strain parallel to Axis of Post & 12 from Axis "	36000 "
& A Horizontal Concentrated Force "	12700
These Load cause Points of Application Result be treated as Moving Loads	
Horizontal Girders IK, G.G. & HK, sustain at centres a Concentrated Horizontal Force "	12700

Note

Girders supporting Sheaves should have 20 per cent allowance for impact

Posts P.P.P.P. may be designed for a maximum Fibre Strain of 15000 lbs

The Above Loads include dead Weights due to Force & Weights of Elevator Machinery and do not include the Weights of the Girders & Posts themselves, which must be allowed for.

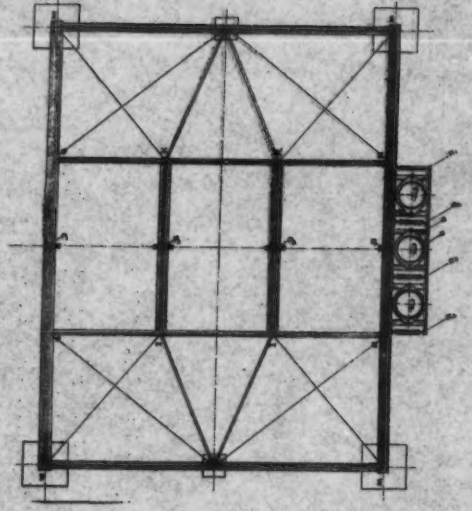


PLATE VII
TRANS: AM. SOC. CIV. ENG'RS
VOL. XXVII N° 535
BROWN ON
WEEHAWKEN ELEVATORS.

1

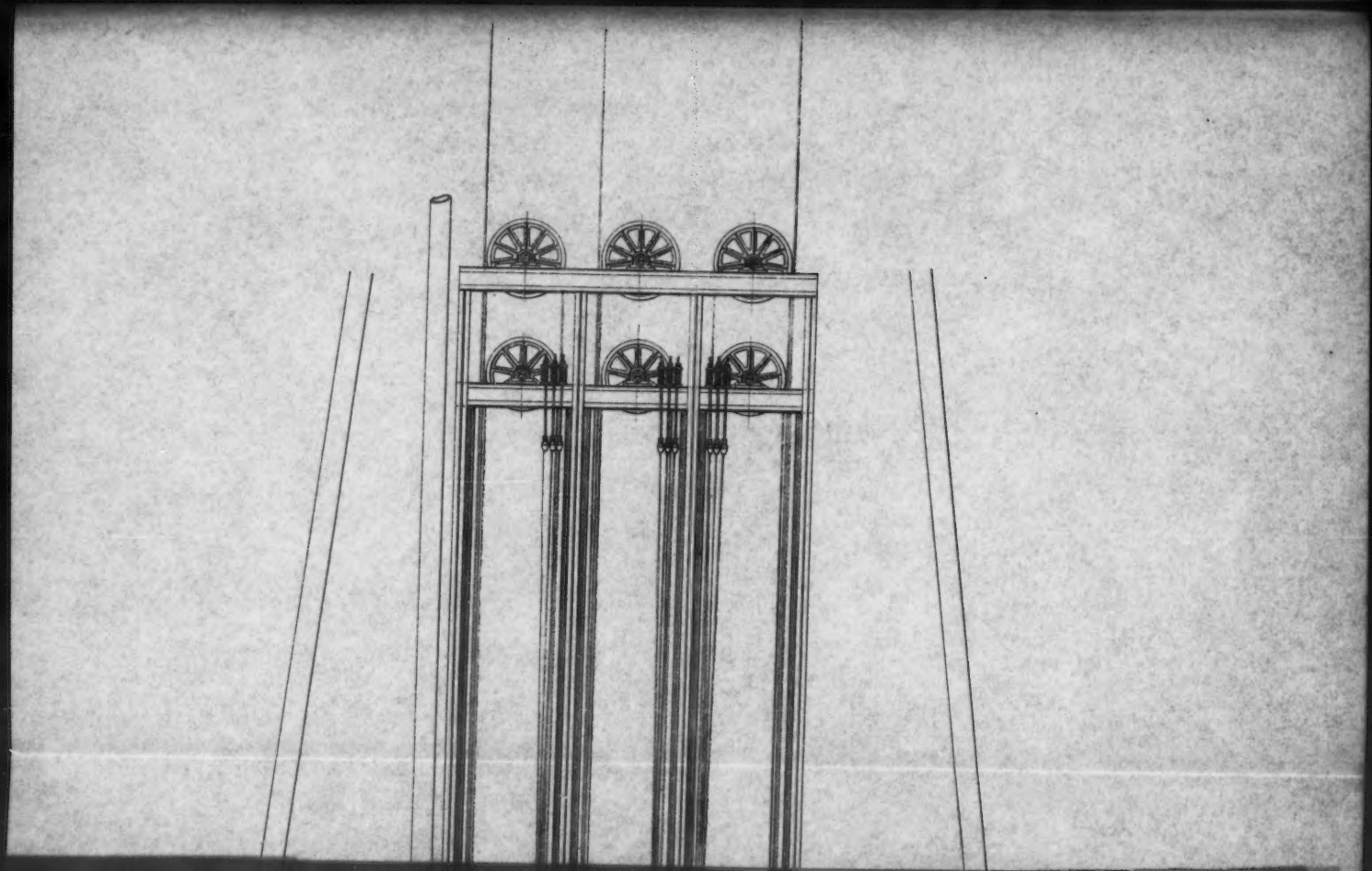


PLATE VIII

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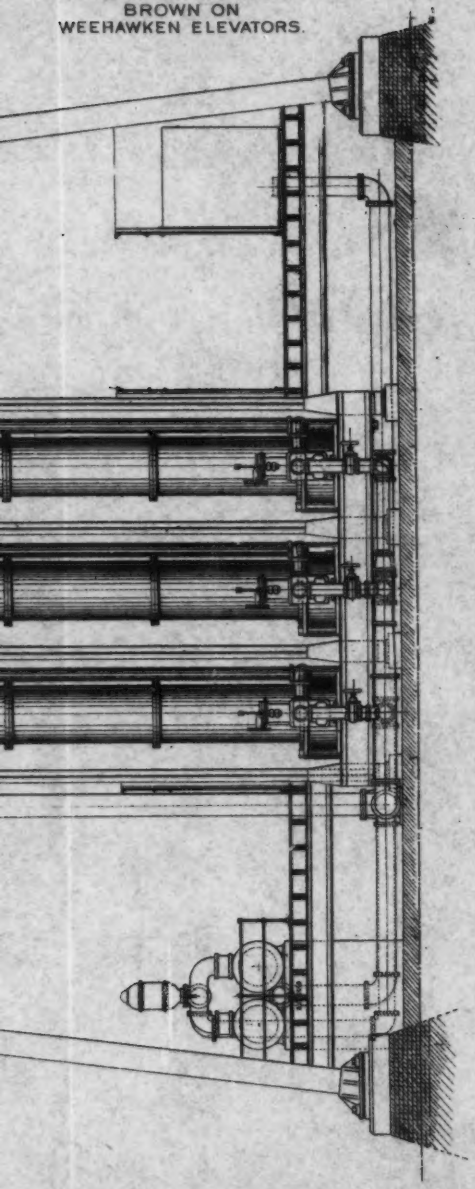
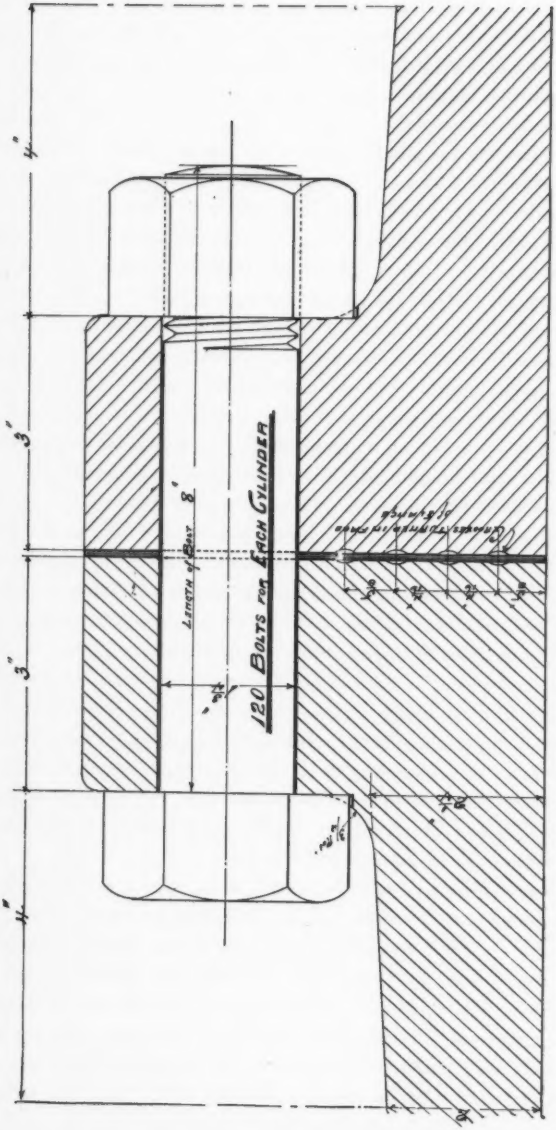
PLATE VIII
TRANS. AM. SOC. CIV. ENGRS
VOL. XXVII Nº 535
BROWN ON
WEEHAWKEN ELEVATORS.





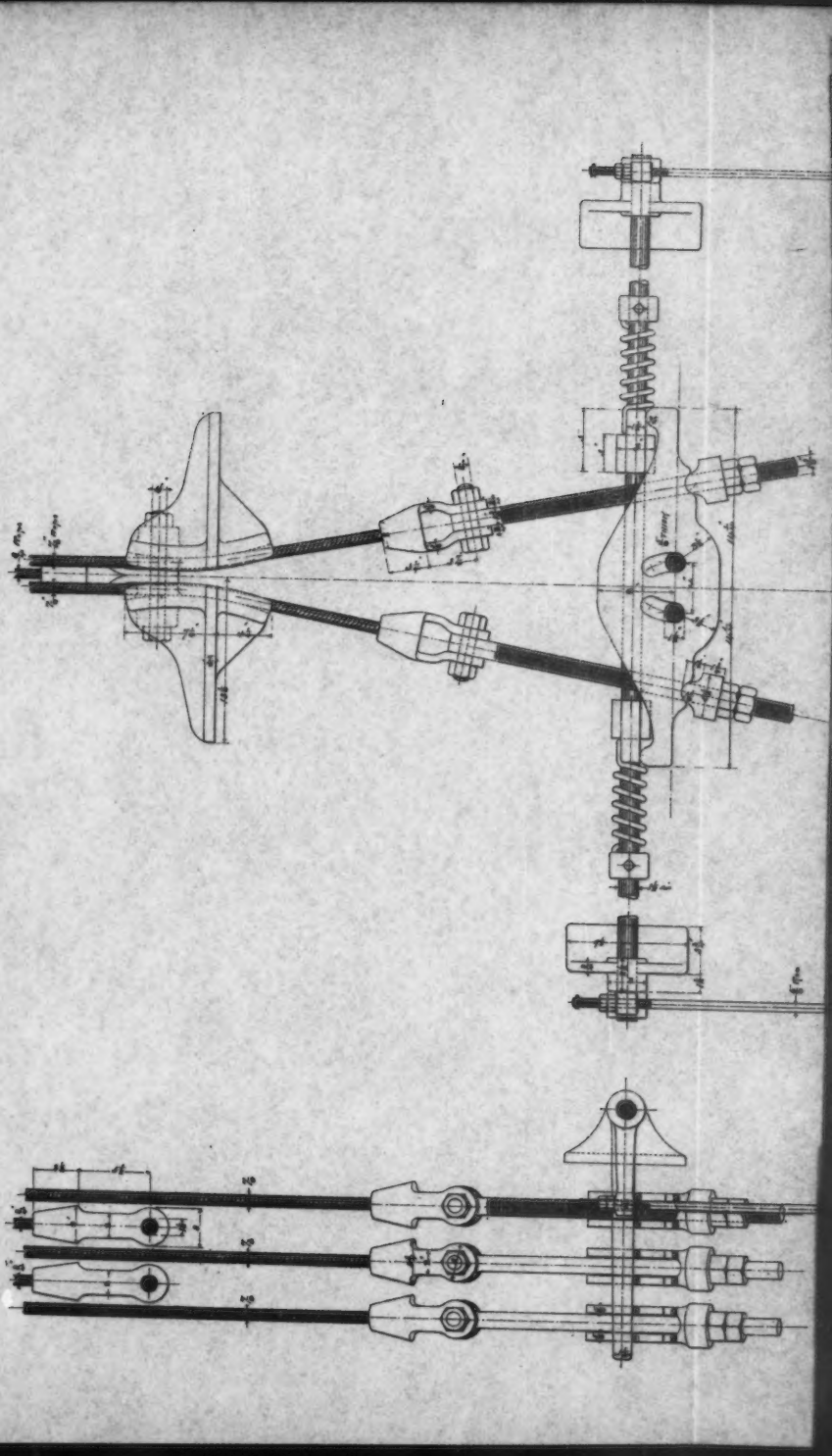
JOINTS IN WATER CYLINDER.

into eyes and connected to equalizing forgings by 4-inch pins, and the equalizing forgings are in turn connected to the wrought-iron traveling-sheave frame by a 5½-inch pin. The strain on the rods is equalized by these equalizing forgings. Each car is suspended by eight crucible steel cables, seven-eighths of an inch in diameter, of six strands of nineteen wires to the strand, with hemp cores. Each cable has an ultimate strength of about 50 000 pounds. Six of the cables are attached to the hoisting machinery, and two to an independent counterbalance weight. Each machine has capacity to lift a load of 20 000 pounds, exclusive of the car at the rate of 200 feet per minute, with a water-pressure of about 185 pounds per square inch.

The cars are provided with Otis Bros. & Co.'s triple grip safety. Six of the steel cables are attached to the girdle frame of the car by tumblers having double centers such that, should any one or more of the cables break or become abnormally stretched, the safety devices will be thrown into action; gripping firmly all three sides of the hardwood guides, and bringing the car and its load to rest gradually, and preventing its further descent. The car may be raised again by the remaining cables, if any, to any position necessary to discharge the load. The tumblers are arranged with double centers to prevent the annoyance of frequent and unnecessary action of the safeties, due to ordinary inequalities of strain and stretch in the ropes (Plate IX).

The cars are also equipped with the "Otis Governor," two to each car. This consists of an ordinary ball governor caused to revolve by a rope attached to the safety devices on the car. The governor operates a grip which, when thrown in by the spreading of the governor balls, grips the rope, thus operating the safety devices and stopping the car. The governors are provided in order that the car may be stopped should it descend at an unsafe speed, from carelessness or other cause, whether the cables are broken or not.

The safety grips were carefully tested at the works of Otis Brothers & Co., first, by pulling sections of the guide strips through them with a hydraulic cylinder, the pressure in which was determined by a gauge. These tests gave the power of a pair of safety clutches as from 50 000 to 53 000 pounds. A practical test was also made by dropping an improvised car loaded with cast iron. The improvised car and its load weighed together 34 922 pounds. It was allowed to drop freely 10¼ inches or until it had acquired a velocity of $7\frac{47}{100}$ feet per second,



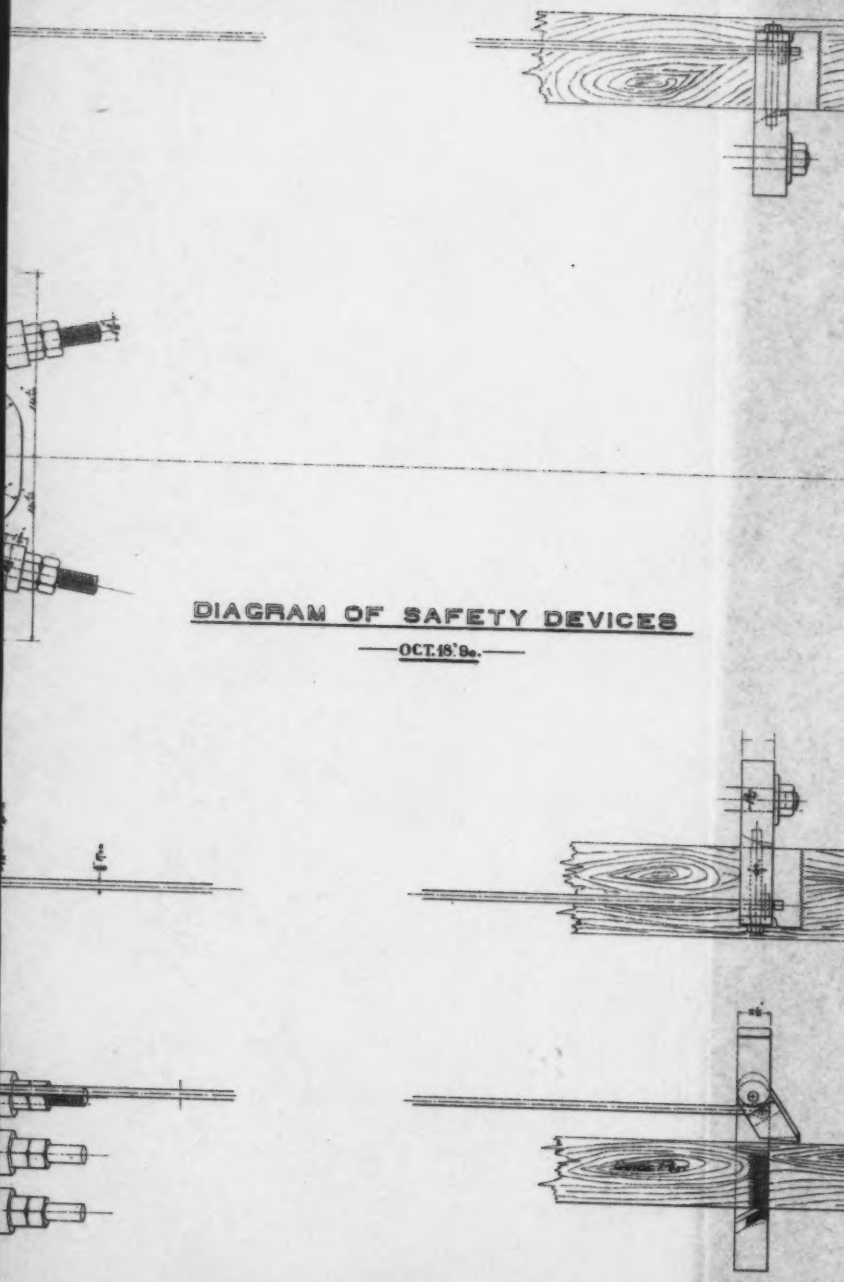
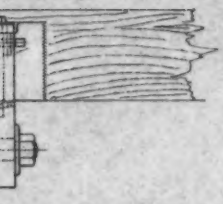
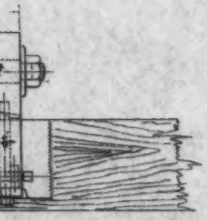


DIAGRAM OF SAFETY DEVICES

OCT. 15. '80.

PLATE IX
TRANS. AM. SOC. CIV. ENGRS
VOL. XXVII Nº 535
BROWN ON
WEEHAWKEN ELEVATORS.





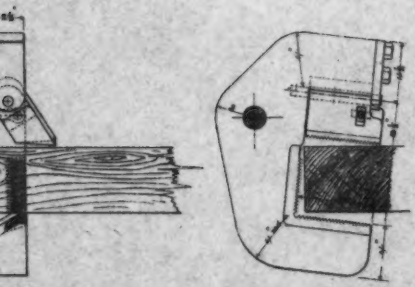


PLATE X.
TRANS. AM. SOC. CIV. ENGS.
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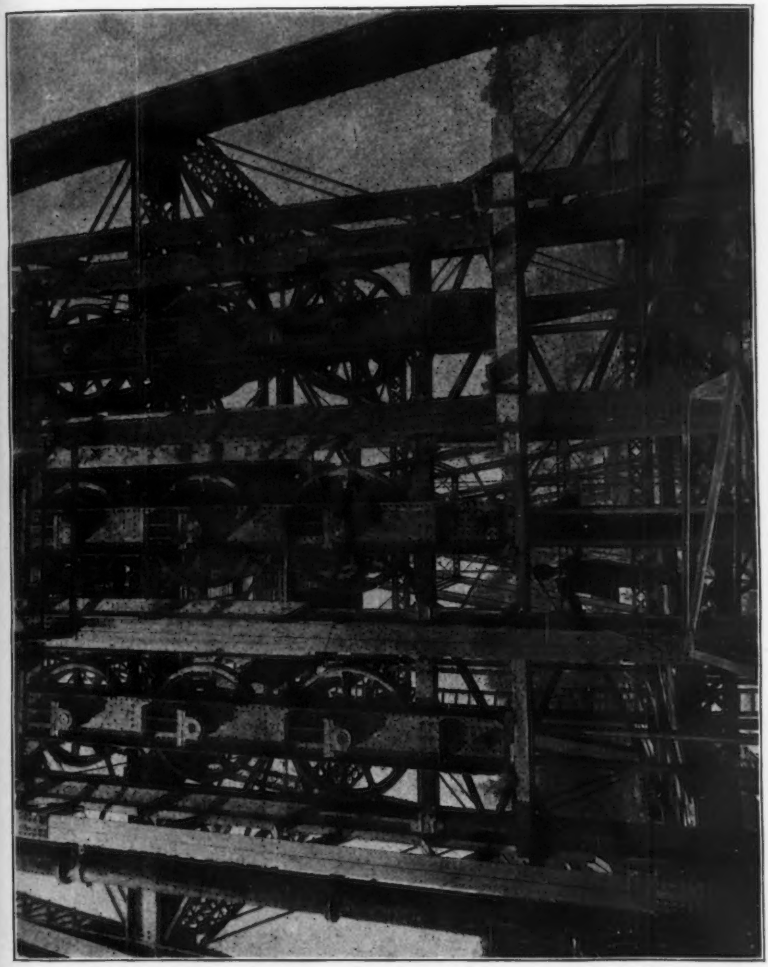
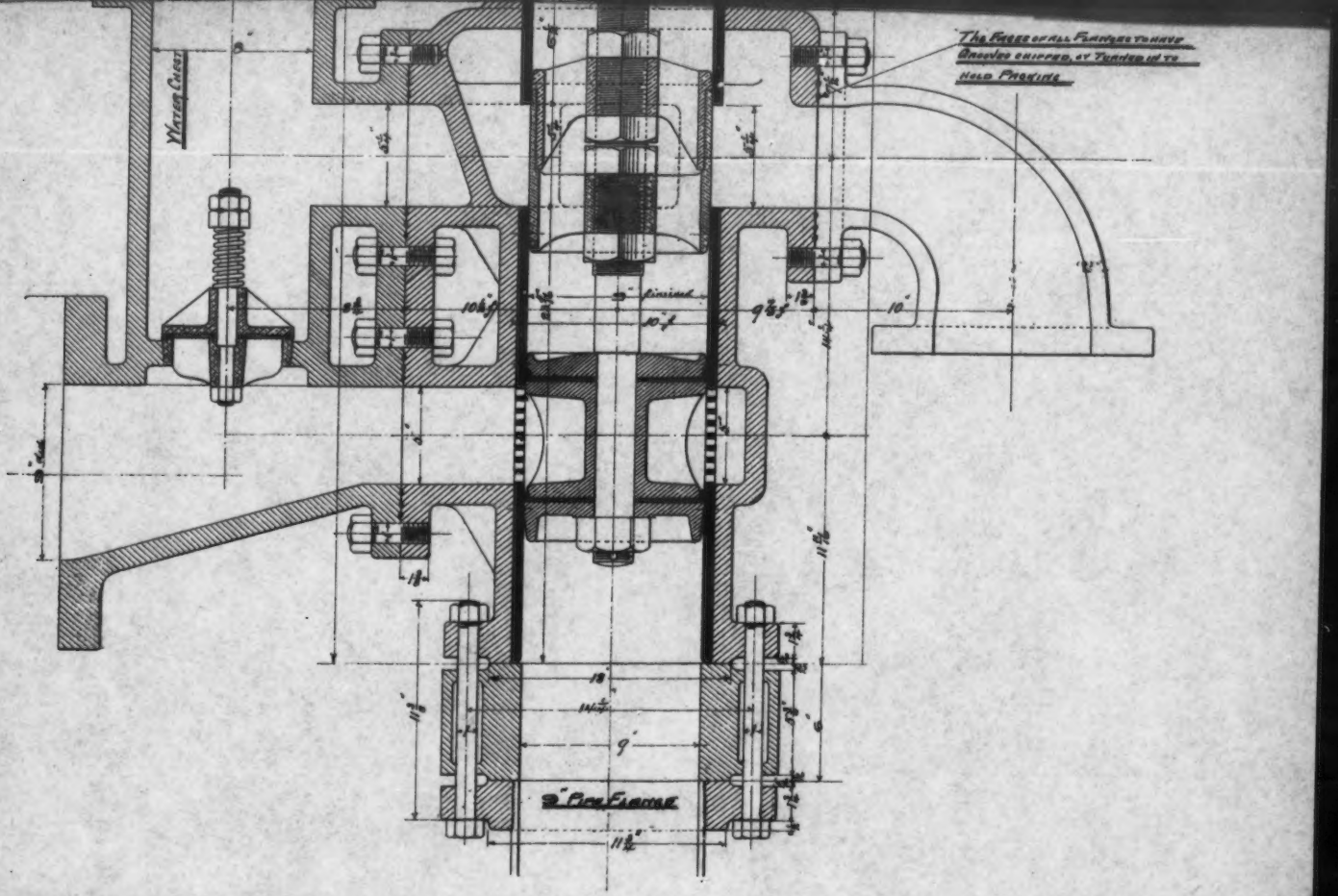
BROWN ON WEEHAWKEN ELEVATORS.




The Faces of all Flanges to have Shoulds Chipped, or Turned in to Hold Packing
Water Chest
10½"
finished
10"
9"
2" Pipe Flange

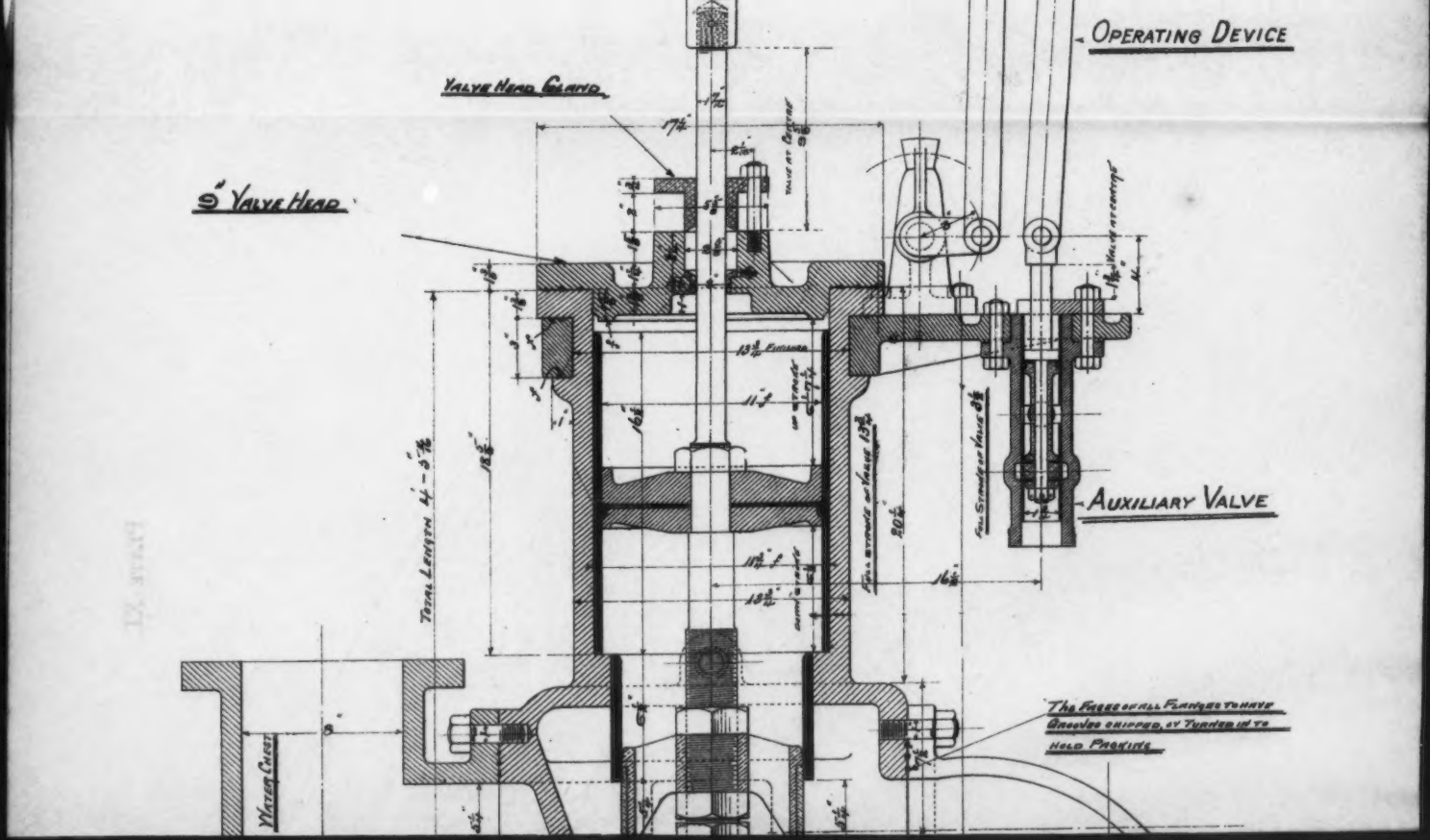

9" Valve Head
Valve Head Gland
Operating Device
Auxiliary Valve
Total Length 4'-5 7/8"
Water Chest
The Faces of all Flanges to have Grooves chipped, or Turned in to hold Packing

PLATE XI
TRANS. AM. SOC. CIV. ENG'RS
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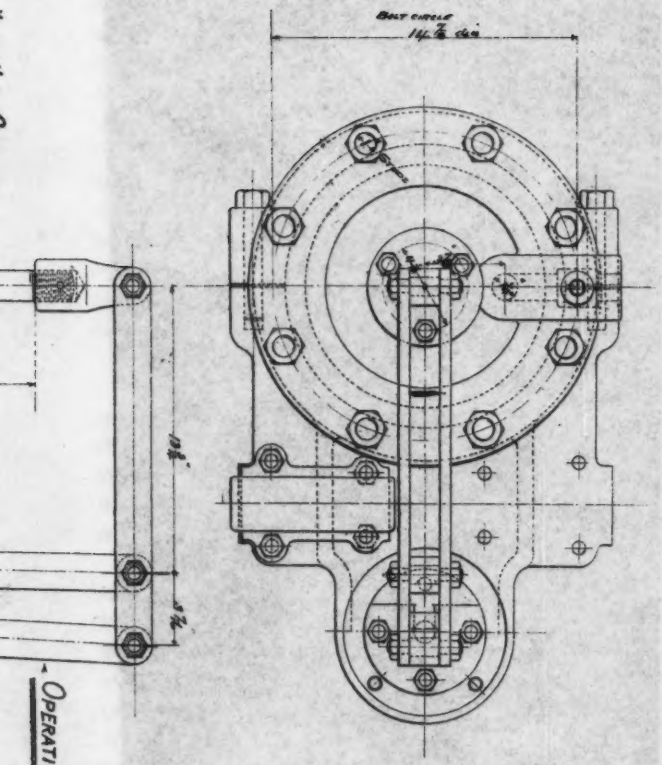
BROWN ON
WEEHAWKEN ELEVATORS.



OPERATING DEVICE

the safety clutches were then tripped, after which the car descended 20¾ inches gradually, and came to rest; from which the resistance of the pair of clutches is shown to be 52 140 pounds, agreeing very closely with the results of the previous tests.

Each set of traveling sheaves consists of three heavy cast-iron sheaves, 60, 56 and 52 inches in diameter respectively; with 6-inch steel shafts, with 8-inch bearing on each side, running in babbitted boxes (Plate X). The sheaves are placed "tandem," and their boxes are supported in a wrought-iron riveted frame. This frame is so designed as to be strained not exceeding 9 000 pounds per square inch of net section. The frame is guided between hardwood guides. The main sheaves are of cast iron 60 inches in diameter, with 6-inch diameter steel shafts, with 8-inch bearings on each side. The boxes are babbitted and are of the 45-degree pattern, with strong sole plates and butt blocks to take the horizontal thrust. The auxiliary sheaves are of the same pattern as the traveling sheaves, with same size axles and boxes; their diameters are 58 and 54 inches respectively. The counterbalance sheaves are 54 inches in diameter and have 4-inch steel shafts, with 5-inch bearings on each side, running in 45 degrees babbitted boxes.

Each car is counterbalanced, as far as practicable, by weights on the piston and in the traveling-sheave frame. The piston, piston rods, traveling-sheave frame, and sheaves serving as counterbalance to the extent of their weight. The remaining counterweight is in an independent frame, running in hard-wood guides. The amount of independent counterbalance is kept as small as possible—an arrangement necessary for smooth stopping and starting.

The operating valve is a differential piston valve, operated by an auxiliary pilot valve, so arranged that the motion of the main valve is proportional to and controlled by the auxiliary valve (Plate XI). The valve piston is 9 inches and the actuating piston 11 inches in diameter. The valve is provided with a secondary adjustable throttling piston to produce gradual closing, and to regulate the speed of descent of the car. The valve is lined with heavy brass, the water passing through perforations in the brass lining to prevent ingress of foreign matter and to reduce the wear of the valve leathers. The valve is provided with two relief valves to prevent shock and danger from water ram, in the event of accidental too rapid closing of the main valve.

The auxiliary pilot valve is used to avoid the severe labor involved in moving the main valves by direct hand-power which is impracticable with as large valves as are required in this case. The auxiliary valve is controlled by a wheel in the car.

The main pressure tank is of 10 000 gallons capacity and is placed in the top of the tower; it is 6 feet 6 inches internal diameter and about 40 feet length of shell with spherical segmental heads. The shell is of steel one-half inch in thickness, and heads five-eighths inch in thickness. The tank is supported in cast-iron saddles which rest upon the upper girders of the tower. The auxiliary pressure-tank, or air chamber, has 1 200 gallons capacity, and is placed at the bottom of the stand-pipe; its purpose is to prevent shock from the working of the pumps, and particularly to prevent water-ram and consequent excessive pressure. It is 42 inches internal diameter and 15 feet length of shell, with shell one-half inch thick, and segmental head five-eighths inch thick. The discharge tank is situated in the cellar or pit under the station floor, and is of 10 000 gallons capacity, and is rectangular in form. It is of one-quarter-inch boiler iron.

The pressure is produced by two Worthington steam pumping engines, of the compound duplex type (with steam cylinders 16 and 29 inches and water cylinders 12 inches in diameter by 18-inch stroke). They are of capacity to run the three elevators on a four-minute schedule (*i. e.*, an elevator starting every ninety seconds), at a piston speed of 50 feet per minute per plunger, with 75 pounds effective steam pressure. By running at a higher speed, one pump alone is sufficient for ordinary service, and, in case of emergency, can be made to do the full service, in the event of the breaking down or disabling of the other. The pumps are provided with an air-pumping device, by means of which air may be supplied at will to either the main or the auxiliary pressure-tanks, or both. The stand-pipe is of cast iron, 15-inch internal diameter, and 1 inch thick; the flanges are faced and packed with rubber gaskets, each joint is bolted with sixteen bolts seven-eighths of an inch in diameter; a stuffing-box at the top provides for expansion. The delivery-pipe from pumps is 12 inches in diameter, and the suction-pipes are 15 inches in diameter, and are provided with foot-valves and strainers in the discharge-tank.

The supply-pipe to each cylinder is 8 inches, and the discharge 9 inches in diameter, and of heavy, wrought-iron pipe. They are pro-

vided with gate-valves so that any elevator may be cut off at will (either for laying up or for inspection or repairs) without interfering with the operation of the others. The pumps are provided with gate-valves on both suctions and deliveries, so that either or both pumps may be used as desired, and are also provided with automatic, steam-regulating valves, actuated by fluctuations in the water-pressure, making the movement of the pumps entirely automatic.

There are three cylindrical boilers, each with two internal corrugated furnaces and return fire tubes, designed for a working pressure of 120 pounds. These boilers are of the following dimensions:

Diameter of shell, 9 feet; length of shell over all, 12 feet; inside diameter of furnace flues, 3 feet; length of furnace flues, about 9 feet 9 inches; length of grate, 6 feet; depth of back connection, inside, 21 inches; thickness of water-back, outside of sheets, 6 inches; one hundred and sixty-nine $2\frac{1}{2}$-inch tubes, about 9 feet 9 inches long; outside diameter of steam dome, 3 feet; height of steam dome over all, 3 feet; heating surface, about 1 500 square feet; grate surface, about 36 square feet; shell, $\frac{11}{16}$ of an inch thick; heads, $\frac{9}{16}$ of an inch thick; flues, three-eighths of an inch thick; furnaces, sides and back of back connection, $\frac{7}{16}$ of an inch thick; tube-sheet of back connection, $\frac{9}{16}$ of an inch thick; shell of steam dome, one-half of an inch. The boilers are covered with 3 inches of magnesia boiler covering, with Russia iron outside. The material used in these boilers is open-hearth steel of a tensile strength of from 60 000 to 66 000 pounds; elastic limit, 36 000 pounds. The stack is 47 inches inside diameter, and its top is about 95 feet above the grates.

There are two condensers of the Wainwright pattern, of 120 nominal horse-power each; each has an internal diameter of shell of 24 inches and contains two hundred and thirty-two half-inch corrugated tubes. The condensed water which is returned to the boilers, and the condensing water which is pumped from the river, are handled by two Knowles' combined air and circulating pumps having steam cylinders 8 inches, air-pump cylinder 9 inches, circulating cylinder 9 inches in diameter respectively, and of 8-inch stroke.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

536.

(Vol. XXVII.—July, 1892.)

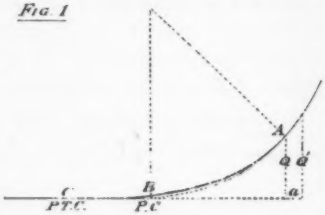
A SIMPLE METHOD OF RUNNING IN A TRANSITION CURVE.

By JOHN F. WARD, M. Am. Soc. C. E.

As train speeds of 100 feet per second are to be expected and must be provided for on all first-class railroads, the necessity of some form of transition curve to modify the shock of entering on a circular curve, has made itself so apparent that a good degree of attention has been directed to the proper shaping of these easements. The cubic parabola has come to be the favorite so far, and in spite of the complexity of some of the investigations of it which have been put forth, it is, doubtless, the best curve for the purpose yet discussed.

In applying the cubic parabola, the simplest and most often occurring problem, is to "put in a transition curve in existing track without running in the whole curve." The first consideration is, how much of the old curve is it desirable to modify, or, to refer to Fig. 1, where shall the point *A* at which the cubic and circular curves join, be placed? The angular distance on the arc from *P. C.* to *A*, should not be more than 20 degrees or thereabout, as the cubic curve departs so much from the circular form when continued beyond that limit, that a satisfactory

Fig. 1



connection is not easily gotten. With the quartic curve a good result is practicable up to 60 degrees, but in practice no one will be apt to make the distance from *P. C.* more than 150 feet, which will come within the angular limit, except on street railroads.

Having assumed the position of *A*, make $BC = .49\ AB$, then *AC*, which is to be the length of the transition curve, is almost one and one-half times the length of the circular arc *AB*, which is to be ameliorated, and in practice it is very handy to call it one and a half. This can be proved to be approximately correct by making *Q* and Q^1 ordinates in the circular arc, at a distance apart equal to *a*. Now, if the curves coincide at *A*, *Q* and Q^1 must also be ordinates in the cubic curve, therefore by the common formula of the curve

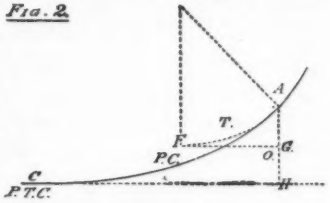
$$\sqrt[3]{Q^1} - \sqrt[3]{Q} : \sqrt[3]{Q} :: a : AC,$$

the length required.

The length *AC* and the *P. T. C.*, or point of transition curve being found, set up the transit on *P. T. C.*, measure the angular distance of *A* from the tangent, and divide it by the square of the number of stations in *AC*, and deflect one of the divisions at the first station, four at the second, and so on, increasing as the square of the distance from the *P. T. C.* If ordinates are used in laying out the cubic curve, divide the length of the offset at the connecting point *A* by the cube of the number of stations, and use one of the divisions for the offset at Station 1, eight for Station 2, etc., the ordinates increasing in length in proportion to the cube of the distance, in the ratio of 1, 8, 27, 64, 125, 216, etc.

Where transition curves are used on new work, the tangent is extended as *CH* (Fig. 2), and instead of the circular curves being connected with the tangent directly, the point of curve is offset any desired distance, and the connecting point *A* (Fig. 2) selected as before, the offset *GH* not to exceed ¼ of *AG*. Then $AG : 1.49\ FG :: AH : HC$, the required length of the transition curve, with an offset at *P. C.* equal to *HG*, and a total ordinate *AH*.

Fig. 2.



To simplify the form of the proportion $AG : 1.49\ FG :: AH : HC$, make $n =$ the number of 100-foot stations in the circular curve *AF* (Fig. 2), $D =$ the degree of curvature per 100 feet, in the circular curve, $AG = O$, $AH = O^1$, $FG = T$, $CH = L$.

Our proportion now becomes $O : 1.49\ T :: O^1 : L$, or $\frac{O^1 T\ 1.49}{O} = L$.

By a formula in common use $O = \frac{7}{8} n^2\ D$, and n being $.01\ T$, $n^2 = .0001\ T^2$, and $\frac{7}{8}\ n^2\ D = .0000875\ T^2 D = O$, $\therefore \frac{O^1 T\ 1.49}{.0000875\ T^2 D} = L$, and

$$\frac{O^1 T\ 17\,000}{T^2 D} = \frac{O^1\ 17\,000}{DT} = L.$$

For track layers' use, requiring neither logarithms, square roots, tables nor other tools than are used in their regular work, we now have this

Rule for Laying Out Transition Curves.

The circular curve FA (Fig. 2) and the point of connection A being given, multiply the distance from A to the tangent by 17,000, and divide the product by the degree of the curve and by the length of the circular curve from $P.\ C.$ to A. The quotient is the length of the transition curve measured on the tangent. Divide this distance into as many stations as are required.

Divide the distance from A to the tangent by the number of stations, divide the quotient by the same number, and that quotient again by the same number. Use this last quotient for the offset to the curve at Station 1, eight times the first offset at Station 2, and so on, increasing in the ratio of 1, 8, 27, 64, 125, 216, that is, as the cubes of the Station numbers.

Where (as in Fig. 1) there is no offset at $P.\ C.$, simply make the transition curve AC one-half longer than the circular curve AB, and put in the offsets as before.

(1) *Example.*—It is required to find the length of a transition curve to join a 7-degree curve at 100 feet from $P.\ C.$, the offset at $P.\ C.$ being 4 feet, the offset O^1 at $A = 10$ feet, and T being 100 feet $\left(\frac{O^1\ 17000}{DT} = L\right)$

$$\frac{10 \times 17000}{7 \times 100} = 243.$$

(2) *Example.*—To find the ordinates of a cubic parabola when the length $L = 160$ feet, and the ordinate $O' = 16$ feet, make four stations, then

$$\begin{array}{r} 4)16 \\ 4)\overline{4} \\ 4)\overline{1.00} \\ \overline{.25} = \text{ordinate at Station 1.} \end{array}$$

$$\begin{array}{rrrrr} .25 \times 8 = & 2. & = & \text{“} & \text{“ } 2. \\ .25 \times 27 = & 6.75 & = & \text{“} & \text{“ } 3. \\ .25 \times 64 = & 16. & = & \text{“} & \text{“ } 4. \end{array}$$

(3) *Example.*—To lay out a cubic parabola by angles when the length $L = 160$ feet, and the angle $ACB = 5^\circ\ 20'$ or 320 minutes, set the transit on $P.\ T.\ C.$, make four stations, and deflect

$$\begin{array}{rrrrr} \frac{320'}{4^2} = & 20' & = \text{angle at} & \text{Station} & 1. \\ 20' \times 2^2 = & 1^\circ\ 20' & = \text{“} & \text{“} & 2. \\ 20' \times 3^2 = & 3^\circ & = \text{“} & \text{“} & 3. \\ 20' \times 4^2 = & 5^\circ\ 20' & = \text{“} & \text{“} & 4. \end{array}$$

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537.

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THE HYDROGRAPHY OF THE POTOMAC BASIN.

By CYRUS C. BABB, Jun. Am. Soc. C. E.

WITH DISCUSSION.

A study of the Potomac River Basin has been made by the writer in connection with the work of the Hydrographic Division of the United States Geological Survey. A gauging station was established on May 4th, 1891, at Chain Bridge, on the Potomac, 3 miles above Washington, D. C., for the measurement of the discharge of the river at that place. From that date to the present time three readings a day have been made of the height of the river, thus making it possible to eliminate the effect of the tide, which has a daily range at this place of about 3 feet.

The cable and trolley method of gauging in practice at this station was first used in England in 1880 on the Severn River, and subsequently in the western United States by the Irrigation Division of the Geological Survey. The apparatus consists essentially of a three-wheeled carriage or trolley running upon a main cable, and moved either forward or backward by means of a set of continuous halyards passing through pulleys at uprights on either shore. Over the third wheel of the trolley, which is below the cable, runs a double conducting wire with

the meter attached at one end, the other being on shore at the place from which the work of gauging is done. This conducting cord fulfills two purposes, one for the transmission of the current, and the other for lowering or raising the meter to any desired depth in the cross-section. Soundings are taken with this cord by having it tagged every foot and estimating the tenths.

About 200 feet above the main cable is stretched a stay cable with an auxiliary trolley running upon it, from which passes a bridle wire down stream which is attached to the meter rod. The purpose of this wire is to prevent the meter from swinging down stream under the influence of the current. As the meter descends it, therefore, describes the arc of a circle, but owing to the length of radius the effect is inappreciable. The use of the stay cable is a departure and an improvement over the English method, in which an anchor of 70 pounds' weight was used.

Theoretically the work of gauging by this system can be accomplished by one man, and in fact it has been done in this manner, but the work is too arduous for general practice on a large stream.

For the measurement of velocities the Haskell electric meter is used, having been found to fulfill most satisfactorily the wants of the hydrographers of the Geological Survey. Many improvements upon the old style of the Haskell meter have been suggested by them, and these have been incorporated in the new type of meter for river work.

Up to March 1st, 1892, five direct gaugings had been made on average discharges of the river, with several partial ones on flood discharges, including the great flood of June, 1889, and the spring freshet of April 1st, 1891. These several gaugings for various river heights were plotted upon cross-section paper, with discharges as abscissas and gauge heights as ordinates. Through the points thus obtained as a smooth curve was drawn, and for each tenth of gauge height along this curve the corresponding discharge was taken and tabulated, forming the rating table. Knowing the height of the river then at any single moment, the corresponding discharge can be taken immediately from this table.

River height observations at Chain Bridge have only been made since May 1st, 1891, so that from this record alone, discharges could be computed only for a period of less than one year. The officials of the Washington aqueduct have kept a daily record of the height of the water flowing over their dam at Great Falls, Md., since 1878, and it has been possible to make use of this record in part.

The Potomac River at this place flows through two channels, forming Conn's Island. The dam of the Washington Aqueduct, for the water supply of the City of Washington, as first built, simply extended across the Maryland channel. In 1886 the present dam was finished, raising the crest of the old dam by 15 inches and extending it on this level across Conn's Island and the other channel, to the Virginia shore. The old dam was 1 034 feet long and the extension 1 843 feet, making a total length of 2 877 feet. The records of river heights from 1878 to 1886 are of little value, owing to the unrestricted flow in the Virginia channel, and also to the fact that in times of low discharge, when the water supply of the city was endangered, temporary wing dams were built at the head of Conn's Island in order to divert water into the Maryland channel and thence into the aqueduct. From the records since 1886 daily discharges of the river for this period have been computed.

The formation of the bed of the Potomac River at Chain Bridge is of rock, so that its cross-section, like that at Great Falls, is practically unchangeable from year to year. From the synchronous records of gauge heights at these two stations it has been possible to construct a rating curve of gauge heights, expressing the gauge height at one station in terms of gauge height at the other. This has been rendered possible by the fact that no large tributary enters the Potomac between these two stations, so that the flow is materially unchanged between them. The combination of the above table with that of the discharge at Chain Bridge gives a rating table for the discharge at Great Falls.

The figures herewith submitted are the results of first operations, and as such are subject to change as additional data are collected. They are considered reliable for general engineering work, but a longer study is necessary for close scientific deductions on such subjects as the following: the cause of the difference in discharge of the river for the same gauge height, between the rise and the ebb of a flood height.

The discharges corresponding to the daily mean gauge heights are taken from the rating table and tabulated. These discharges are plotted in order to show graphically the fluctuations of the river from day to day and month to month. They are also averaged by months, as shown in the table on pages 26, 27 and 28, which gives the monthly flows of the Potomac River at Great Falls from 1886 to 1891, inclusive. The first column gives the month; the second, the maximum discharge

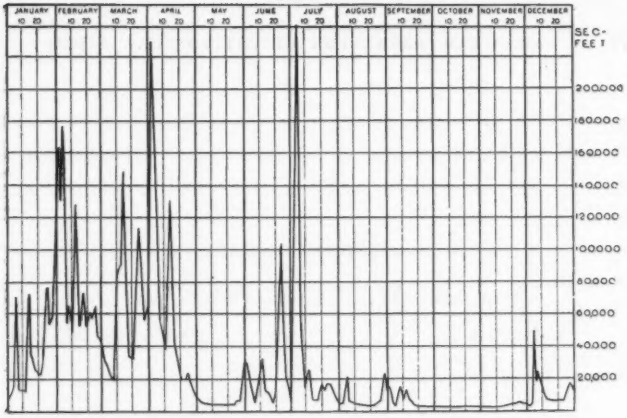
in cubic feet per second, or second-feet as it is commonly called; the third, the minimum; and the fourth, the mean discharge for the month. The last two columns express the flow in two ways in connection with the drainage basin, which at this point has an area of 11 043 square miles. The fifth column gives the depth of run-off, or the depth in inches which a plain of the size of the drainage basin at this point would be covered, if the total flow for the month were spread over it. The sixth and last column gives the second-feet per square mile of area drained. The figures for 1886 are the least reliable of the series, due to the fact that the construction work on the dam was not completed until August of that year.

It is seen from these records that 1887 is the dry year for this period, with a mean monthly discharge of 11 880 second-feet. In 1888 it was higher, with 15 365 second-feet, and the maximum was reached in 1889, with an average discharge of 32 913 second-feet. 1890 was a year of average flow of 21 368 second-feet, and in 1891 it had increased to 26 928 second-feet. The yearly rainfall records show the same general variations.

Plate XII, Fig. 1, is a diagram of the daily discharge of the Potomac River for 1891, and it shows the very erratic nature of this stream. The diagrams for the other years are of a similar character.

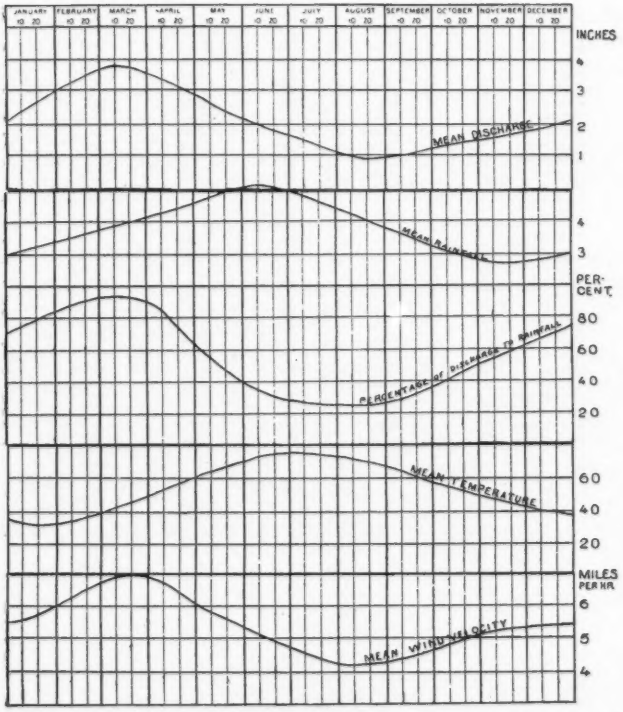
It will be of interest to compare the Potomac discharge diagram with some of those for our western rivers as they appear in the Twelfth Annual Report of the Director of the U. S. Geological Survey. Take, for instance, the Yellowstone at Horr, Montana, as a typical western stream. The first three months of the year show a season of low water. In April the river starts on its spring rise, culminating in the latter part of May. It holds this general level for two or three weeks, with minor fluctuations due principally to variations in temperature, and then gradually declines through the months of July, August and September, with a low-water discharge for the rest of the year. This difference in the character of the two streams is accounted for by the fact that the Yellowstone is a snow-fed river, while the Potomac is a rain-fed one. The Yellowstone at point of gauging has a drainage area of 2 700 square miles, with a total yearly run-off of 14 inches. Compare this with the average yearly run-off of 24 inches of the Potomac River, and consider that the yearly flow of the Yellowstone River is above the average for the majority of our western streams;

FIG. 1.



DAILY DISCHARGE OF THE POTOMAC RIVER
GREAT FALLS. MD 1891.

FIG. 2.



POTOMAC RIVER BASIN.
AVERAGE CURVES FOR A SERIES OF YEARS.
DISCHARGE, RAINFALL, ETC.

TABLE No. 1.

Potomac River at Great Falls, Md.

Drainage Area.....11 043 square miles.

	Month.	Discharge in Second-Feet.			Run-off.	
		Maximum.	Minimum.	Mean.	Depth in inches.	Second-feet per square mile.
1886.	January	154 560	2 620	13 711	1.43	1.24
	February	161 600	2 460	22 523	2.20	2.04
	March	65 400	2 800	5 537	.58	.50
	April	224 600	2 800	44 313	4.46	4.01
	May	167 840	2 620	24 206	2.52	2.19
	June	3 240	2 800	2 993	.30	.27
	July	24 800	2 800	5 280	.55	.48
	August	20 400	3 240	6 010	.63	.54
	September	3 580	3 240	3 387	.34	.31
	October	4 290	3 240	3 318	.35	.30
	November	39 500	3 240	7 948	.80	.72
	December	29 700	3 240	10 903	1.14	.99
	Total	.....	.....	.....	15.30	.....
	Mean	.....	.....	12 511	.....	.....
1887.	January	104 320	4 290	12 208	1.27	1.11
	February	54 200	5 460	24 256	2.28	2.30
	March	84 000	5 460	27 311	2.84	2.48
	April	71 000	4 290	14 113	1.42	1.28
	May	104 320	5 460	25 181	2.62	2.28
	June	29 700	4 290	14 159	1.43	1.28
	July	29 700	3 240	6 118	.64	.55
	August	5 460	3 000	3 541	.37	.32
	September	4 290	3 240	3 581	.37	.32
	October	3 580	3 240	3 349	.35	.30
	November	3 240	3 240	3 240	.33	.29
	December	10 500	3 240	5 613	.58	.51
	Total	.....	.....	.....	14.50	.....
	Mean	.....	.....	11 880	.....	.....

TABLE No. 1—(*Continued*).

POTOMAC RIVER AT GREAT FALLS, MD.

Drainage Area.....11 043 square miles.

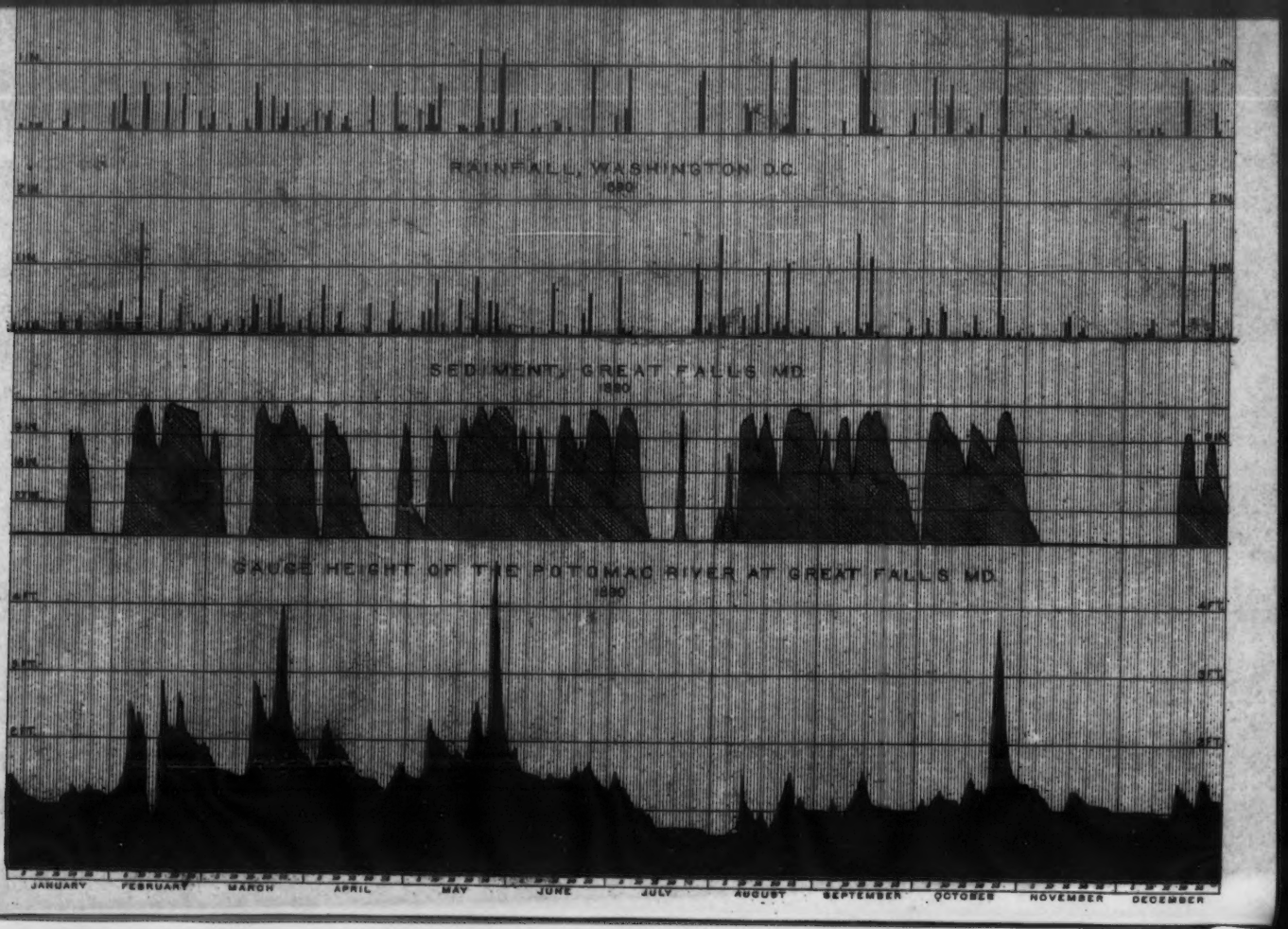
	Month.	Discharge in Second-Feet.			Run-off.	
		Maximum.	Minimum.	Mean.	Depth in inches.	Second-feet per square mile.
1888.	January	44 400	3 240	12 754	1.33	1.15
	February	65 400	4 290	27 768	2.70	2.51
	March	59 800	7 900	28 897	3.01	2.62
	April	44 400	4 290	17 990	1.82	1.63
	May	29 700	3 580	10 634	1.11	.96
	June	34 600	3 000	8 707	.88	.79
	July	213 000	3 240	18 640	1.94	1.69
	August	24 800	3 000	4 942	.51	.45
	September	84 000	3 240	16 833	1.70	1.52
	October	39 500	3 580	7 077	.74	.64
	November	54 200	5 460	16 435	1.66	1.49
	December	84 000	3 580	13 703	1.43	1.24
	Total	.....	.....	.....	18.83	.....
	Mean	.....	.....	15 365	.....	.....
1889.	January	120 080	5 460	31 292	3.26	2.83
	February	65 400	5 460	19 049	1.77	1.72
	March	188 200	10 500	37 932	3.95	3.43
	April	167 840	7 900	35 791	3.61	3.24
	May	44 400	7 900	19 020	1.98	1.72
	June	471 700	3 580	47 761	4.65	4.32
	July	65 400	3 240	9 610	1.00	.87
	August	65 400	5 460	16 393	1.71	1.48
	September	71 000	5 460	25 497	2.57	2.31
	October	137 680	7 900	29 573	3.08	2.67
	November	167 840	20 400	65 214	6.58	5.90
	December	104 320	16 300	27 820	2.90	2.52
	Total	.....	.....	.....	37.06	.....
	Mean	.....	.....	32 913	.....	.....

TABLE No. 1—(*Continued*).

POTOMAC RIVER AT GREAT FALLS, MD.

Drainage Area.....11 043 square miles.

	Month.	Discharge in Second-Feet.			Run-off.	
		Maximum.	Minimum.	Mean.	Depth in inches.	Second-feet per square mile.
1890.	January	16 300	5 460	9 652	1.01	.87
	February	84 000	3 580	38 948	3.68	3.52
	March	180 000	16 300	48 920	5.09	4.43
	April	59 800	10 500	25 330	2.56	2.29
	May	204 700	16 300	50 422	5.26	4.56
	June	34 600	13 100	20 553	2.07	1.86
	July	24 800	3 000	6 426	.67	.58
	August	24 800	3 000	6 748	.71	.61
	September	24 800	3 580	7 913	.80	.72
	October	154 560	5 460	23 616	2.46	2.14
	November	16 300	5 460	9 394	.94	.85
	December	20 400	4 290	8 457	.88	.77
	Total	.....	.....	.....	26.13	.....
	Mean	.....	.....	21 368	.....	.....
1891.	January	120 080	7 900	40 710	4.25	3.68
	February	173 920	34 600	79 986	7.54	7.23
	March	154 560	20 400	59 653	6.23	5.40
	April	229 600	10 500	71 444	7.22	6.47
	May	29 700	4 290	6 260	.65	.57
	June	104 320	5 460	22 062	2.23	2.00
	July	24 800	5 460	11 717	1.22	1.06
	August	24 800	3 240	7 102	.74	.64
	September	16 300	3 000	6 022	.61	.54
	October	3 580	3 000	3 948	.41	.35
	November	5 460	3 240	4 546	.46	.41
	December	54 200	3 580	9 682	1.01	.88
	Total	.....	.....	.....	32.57	.....
	Mean	.....	.....	26 928	.....	.....


RAINFALL, WASHINGTON D.C.
1930
SEDIMENT, GREAT FALLS MD.
1930
GAUGE HEIGHT OF THE POTOMAC RIVER AT GREAT FALLS MD.
1930
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER

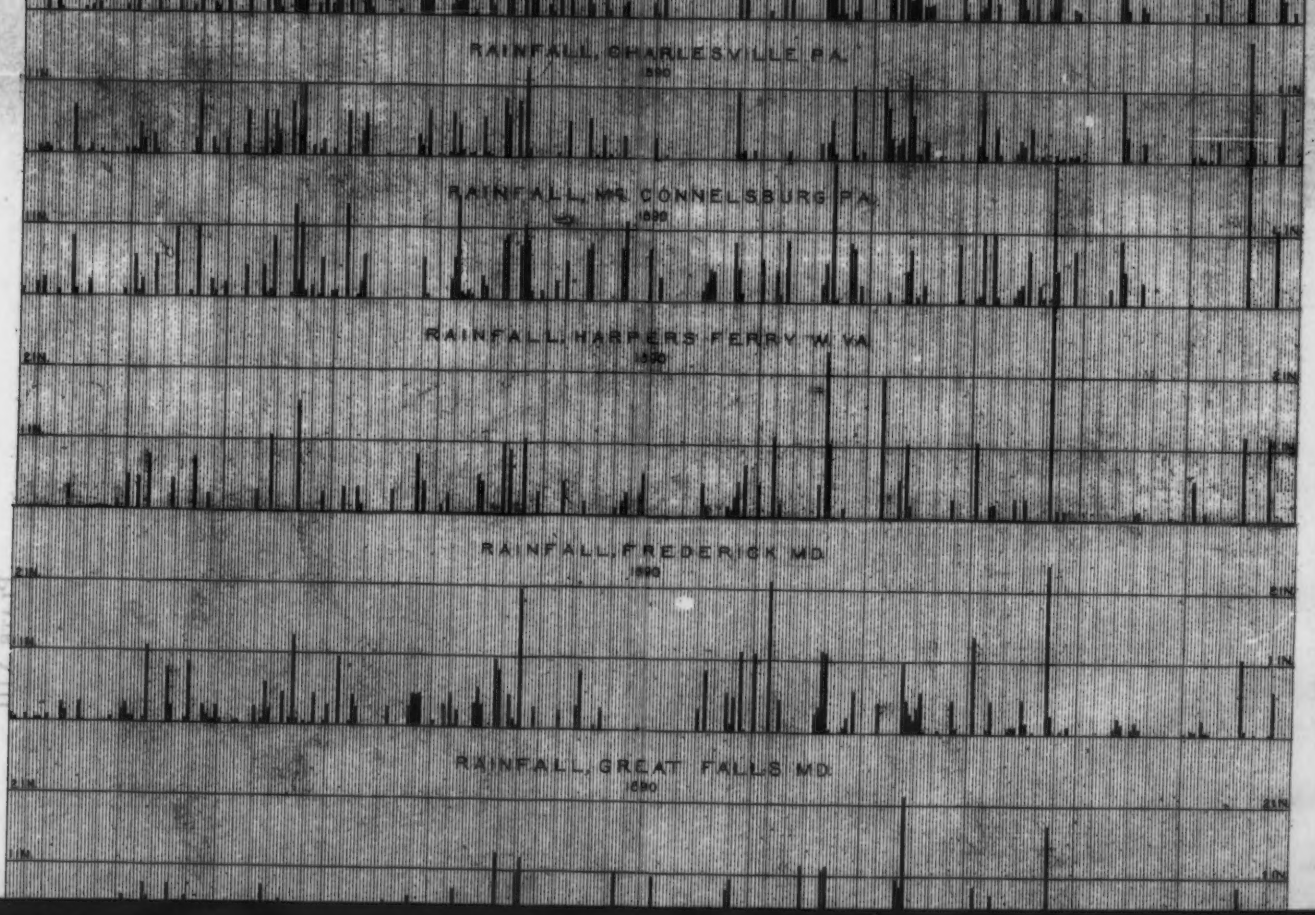

RAINFALL, CHARLESVILLE PA.
1890
RAINFALL, MT. CONNELSBURG PA.
1890
RAINFALL, HARPERS FERRY W. VA.
1890
RAINFALL, FREDERICK MD.
1890
RAINFALL, GREAT FALLS MD.
1890

PLATE XIII.
TRANS.AM.SOC.CIV.ENGRS.
VOL.XXVIII.Nº 537.
BABB ON HYDROGRAPHY OF
THE POTOMAC BASIN.

for instance, the Missouri at Craig, Montana, with a drainage area of 17 615 square miles, has a total yearly run-off of only 4 inches.

Rainfall.

We now come to a subject which in its relation to river-flow makes a very interesting study, but, at the same time, with our present state of knowledge, an unsatisfactory one.

Records of the daily rainfall at various points in the Potomac basin for a series of years have been obtained from the Weather Bureau, and on Plate XIII is shown the daily rainfall at eight stations throughout the basin for the year 1890, with also the daily gauge height of the Potomac River at Great Falls and the curve of sediment observations at the same place. From this diagram the river is seen to respond in a certain degree to a daily rainfall, but owing to the size of the basin the effect of a rainfall in a certain part of it upon the total flow of the river cannot be interpreted until the different peculiarities of the sub-basins have been investigated, and until there has been found the effect a rainfall has on the stream-flow of the sub-basin in which it occurs.

Table No. 2 shows the relation between the monthly rainfall and the run-off. The first column for each year gives the corrected total monthly rainfall, a rain occurring at the end of a month being added to the rainfall for the following month as affecting the river-flow of that month. The second column gives the river-flow in inches, and the third the percentage of river-flow to rainfall.

An inspection of this table will show the futility of computations of discharge based upon percentages of monthly rainfalls. A general rule has not yet been found by which river-flows can be deduced from rainfall records, although the method of Mr. C. C. Vermeule, as presented in his report on the water supply of the State of New Jersey, seems nearer the solution of the question than any heretofore presented. His deductions are based upon the results of work on rivers of small water-sheds, and before they can be applied to the Potomac River the coefficients will have to be modified, as is shown by an off-hand application of the methods. This can only be done after a more detailed study of the basin has been made.

In Table No. 3 appear the averages of the foregoing figures. The second column, headed "smoothed out," under each general subject requires an explanation. They are derived from the figures of the first

TABLE No. 2.

Month.	1886.			1887.			1888.			1889.			1890.			1891.			Average per cent.
	Corrected rainfall. Inches.	River-flow. Inches.	Per cent.	Corrected rainfall. Inches.	River-flow. Inches.	Per cent.	Corrected rainfall. Inches.	River-flow. Inches.	Per cent.	Corrected Rainfall. Inches.	River-flow. Inches.	Per cent.	Corrected rainfall. Inches.	River-flow. Inches.	Per cent.	Corrected rainfall. Inches.	River-flow. Inches.	Per cent.	
January	4.40	1.43	32.5	2.38	1.27	53.4	3.27	1.33	40.7	3.97	3.26	82.2	1.44	1.01	70.3	3.79	4.25	112.1	65.2
February	3.52	2.20	62.4	3.88	2.28	58.7	2.71	2.70	100.0	1.85	1.77	95.6	3.98	3.68	92.4	4.15	7.54	181.8	98.5
March	4.59	.58	12.6	2.86	2.84	100.0	3.68	3.01	81.8	4.37	3.95	90.5	4.16	5.09	122.5	6.09	6.23	102.2	84.9
April	3.81	4.46	117.5	3.56	1.42	40.0	2.01	1.82	90.6	5.14	3.61	70.2	3 11	2.56	82.4	3.23	7.22	223.8	104.1
May	8.61	2.52	25.0	3 51	2.62	74.7	3.71	1.11	30.0	5.15	1.98	38.5	6.86	5.26	76.7	2.80	.65	23.2	44.7
June	6.34	.30	4.7	4.44	1.43	32.6	3.40	.88	25.9	9.55	4.65	48.7	2.92	2.07	70.9	4.83	2.23	46.2	38.2
July	6.81	.55	8.2	5.04	.64	12.7	3.65	1.94	53.3	4.18	1.00	23.9	3.04	.67	22.0	6.61	1.22	18.5	23 1
August	2.58	.63	24.4	2.17	.37	17.1	4.78	.51	10.7	3.83	1.71	44.6	5.49	.71	13.0	4.02	.74	18.4	21.3
September	2.21	.34	15.4	3.58	.37	10.3	5.34	1.70	31.9	4.91	2.57	52.3	4.15	.80	19.8	2.98	.61	20.5	25.0
October	1.60	.35	21.8	1.21	.35	29.0	2.74	.74	27.2	3.70	3.08	83.2	5.59	2.46	44.1	1.09	.41	37.7	40.5
November	4.03	.80	19.9	1.17	.33	28.2	3.09	1.66	53.8	5.95	6.58	110.1	1.04	.94	90.3	1.98	.46	23.2	54.3
December	2.60	1.14	54.5	3.23	.58	18.0	2.60	1 43	55.0	1.20	2.90	241.6	3.14	.88	28.4	2.77	1.01	36.4	72.3
Totals	51.10	15.30	30.0	37.03	14.50	39.2	40.99	18.83	46.0	53.80	37.06	68.9	44.92	26.13	58.2	44.34	32.57	73.4	53.0

TABLE No. 3.

AVERAGES.

MONTH.	DISCHARGE.		RAINFALL.		PER CENT.		TEMPERATURE.		WIND.	
	Average. Inches.	Smoothed out. Inches.	Average. Inches.	Smoothed out. Inches.	Average. Per cent.	Smoothed out. Per cent.	Average. Degrees.	Smoothed out. Degrees.	Average. M. per H.	Smoothed out. M. per H.
January	2.09	2.21	3.21	3.09	65.2	75.3	34.5	33.1	5.4	5.6
February	3.36	3.11	3.35	3.57	98.5	86.8	29.3	34.9	6.0	6.2
March	3.62	3.53	4.39	3.90	84.9	93.1	42.3	41.9	7.4	6.9
April	3.51	3.25	3.48	4.12	104.1	84.4	53.8	53.5	6.7	6.7
May	2.36	2.54	5.11	4.74	44.7	57 9	64 1	64.3	5.4	5.6
June	1.93	1.81	5.25	5.12	38.2	36.5	75.1	72.3	5.0	5.1
July	1.00	1.18	4.89	4.71	23.1	26.4	75.1	74 2	4.5	4.5
August	.78	.91	3.81	4.09	21.3	22.7	71.7	70.9	4.0	4.2
September	1.06	1.03	3.86	3.54	25.0	27.9	64.9	63.4	4.4	4.4
October	1.21	1.32	2.65	3.01	40.5	40.1	52.0	53.6	4.9	4.9
November	1.79	1 53	2.88	2.75	54.3	55.3	45.6	46.8	5.3	4.8
December	1.32	1.63	2.59	2 82	72.3	66.0	44 1	42.1	5.7	5.4
Total	24.03	.....	45.47							
Mean	.....	.....	.....	.....	53.0	.....	54.4	.....	5.4	

column in each case, by applying to the latter the formula $\frac{1}{4}(a+2b+c)$, in which "b" represents the figures in the first column for the month under consideration "a," the figures for the preceding month, and "c," those for the following month, as for instance, under discharge for the month of February we find 3.11 inches in the "smoothed out" column. It is derived as follows: $\frac{1}{4}(2.09+2\times 3.36+3.62)=3.11$. This method eliminates abnormal observations, and where the resultant figures are treated graphically a more characteristic curve is the result. On Plate XII, Fig. 2, are shown these smoothed-out values. At the top of the diagram is the normal curve of the monthly flow of the river; below that is the normal monthly rainfall curve, and then the curve showing the relation between these two, or the percentage of discharge to rainfall. The two lower curves are those of temperature and wind movement. No direct connection between the river-flow and rainfall curves can be perceived on this diagram. The maximum point for the former occurs in March, and the minimum in August, while for the latter curve the corresponding points are not reached until June and December. The comparison of the third or ratio curve to the upper two is of interest as showing the relation between the three. It is seen to follow

quite closely the discharge curve. No especial inferences are to be drawn from the two lower curves. They are inserted as being of general interest.

The Potomac River, formed by the junction of its two main affluents, the North and South branches, takes its rise in the Allegheny Mountains, the North branch in the western part of the State of Maryland, and the South branch in the State of West Virginia. The direction of flow is to the northeast, draining generally narrow and precipitous valleys. After the union of the two branches the main river has a direction nearly at right angles to its previous course and to the trend of the mountains.

The drainage areas of the North and South branches at their junction are 1 365 and 1 407 square miles respectively; of the main river at Harper's Ferry, above its junction with the Shenandoah, 6 354 square miles, and of the latter stream at the same point, 3 009 square miles.

The Potomac is subject to very sudden and heavy freshets, due to its narrow and steep-sided valleys, and also to the fact of the non-occurrence of lakes throughout its basin, which would act as equalizers to the flow, reducing the intensity of the floods and increasing the summer flow.

The slope of the South branch from a point 10 miles above Moorfield to its mouth is 5.7 feet per mile; of the Shenandoah from Port Republic to Front Royal it is 6.0 feet per mile; from Front Royal to its mouth the fall is 3.9 feet per mile; and, finally, from Port Republic to the mouth, 5.3 feet per mile. It is thus seen that for their entire courses the South branch and the Shenandoah River have nearly the same fall, but owing to the narrower and steeper-sided valleys of the former stream, and to the phenomena of subterranean flow in the Shenandoah basin, the South branch is subject to the greater variations in discharge. The fall of the main river between Cumberland and Harper's Ferry is 3.7 feet per mile; between Harper's Ferry and Washington, 4.0 feet, or a slope of 2.5 feet per mile if the fall of 90 feet occurring at Great Falls be subtracted. For the entire length from Cumberland to Washington the fall is 3.3 feet per mile.

The tributaries of the Potomac entering from the north do not drain such an extent of country as the southern ones. Their basins are of a more rolling character, the declivities are not as great, and the general altitudes of the areas are less. The rivers are, however, liable to quite violent freshets.

A word now in regard to the sediment curve represented upon Plate XIII. Simultaneously with the gauge-height records of Great Falls made by the officials of the Washington Aqueduct, another record has been kept, namely, "condition of water," as it is called, or observations upon the amount of matter carried in suspension in the river. The method of measurement is as follows: A horizontal metallic tube, 36 inches long and with glass ends, is filled with water, and the distance at which a ball immersed therein can be seen from one end is noted. The observations vary from 1 inch in very muddy water to 36 inches, which is considered clear.

Besides the measurement of the discharge of the river, the writer has caused to be collected samples of the river water at Great Falls. These are sent into the main office, where they are analyzed quantitatively, in so far as the determination of the ratio of the weight of the contained sediment to the total weight of the sample. From these measurements it is hoped that a rating curve may be constructed for the record of "condition of water," interpreting these visual and relative records into quantitative values. A great degree of accuracy is not expected to be found in the results, owing to the crude method of measurement in use by the officials of the Washington Aqueduct, but it is hoped that from the general law of averages the computations for a series of years will approximate the truth. The quantitative measurements have not as yet progressed far enough for computations to be made. The sediment curve shown on Plate XIII is a graphical representation of the Washington Aqueduct record for 1890 of "condition of water."

DISCUSSION.

G. H. MENDELL, M. Am. Soc. C. E.—In California there is a mountainous area of about 12 square miles, the drainage from which is either mainly or entirely stored in reservoirs. The yearly rainfall is subject to great variation, being as little as 20 inches and as much as 80 inches. A moderate season gives about 40 inches. The capacities of the reservoirs are sufficient, in general, for storage of this water, but, exceptionally, some has to be wasted. The rainy season may be stated as extending from October to April, although a little rain falls in other months. The summer and autumn make the dry season. The num-

ber of rainy days in a season is usually found between the limits of forty and ninety days. The season usually consists of several days of rain, followed by a period of clear weather. This circumstance permits the drainage of each storm, for a period of days, to be measured. The rainfall over the basin, or rather, basins, for there are two (the overflow from one reservoir being conducted to the other), is taken to be as recorded respectively in two gauges, one at each reservoir. The ground above the reservoirs is high, and generally wooded; and the declivities are steep, but not generally rocky. Observations of rain and reservoir gauges, kept during a number of years, offer an opportunity to study the delivery to the reservoirs as to rate and quantity.

Each season has its own law of productiveness dependent, not only upon the amount, but also upon the mode of occurrence of rainfall. Eight inches have fallen in twenty-four hours, and, in some months, 25 or more inches, giving great monthly inequalities in the same season. In other years the monthly rates are more nearly equal.

In spite, however, of the rule of extreme variations in quantities and modes of occurrence, certain generalizations of more or less definiteness may be drawn from statistics covering a number of years, and including all kinds of seasons, excessive, moderate and dry, so-called.

The ground, at the beginning of the wet season, is in condition to take up moisture readily, and the first 10 inches of rain mainly, if not entirely, go to the ground and foliage. In moderate seasons 25 to 30 per cent. of the total rainfall runs off. In seasons of exceptional severity, the reservoirs are not able to take all the water. The waste has not been measured, but the drainage exceeds 40 per cent.

As the season advances, and the ground approaches more nearly to saturation, a given rainfall yields an increased percentage of product. In one case, a month yielded for storage more than 60 per cent. of the corresponding rainfall. The first 20 inches generally give a run-off of 10 or 12 per cent. If, however, the total rainfall of the season does not much exceed 20 inches, and is distributed through the winter and spring, the product is very little, and not far from 5 per cent.

Generally, in a moderate season, the first 30 inches give a run-off about 15 to 20 per cent. In one case, where 30 inches fell in the early part of the season, in a short time, as much as 30 per cent. ran off. After 30 inches have fallen, the product of the remainder of the season is generally found between the limits of 45 and 55 per cent. of the corresponding rainfall. But, in case of exceptional rainfall, the proportion of drainage may be greater.

There is a useful lesson in the fact that the proportion of run-off in the latter part of a good season is large. When waste-ways are not

proportioned to carry this excessive flowage, it is plain that safety of the earthern embankment imposes great caution in filling the reservoir. There is temptation to this policy in countries where rainfall is uncertain and often insufficient.

H. F. DUNHAM, M. Am. Soc. C. E.—The presence of steep-sided valleys and absence of lakes in the drainage area of the Potomac are undoubtedly due to the fact that the whole district referred to in Mr. Babb's paper is south of the area covered by glaciers during the ice age. In Virginia, no deeply eroded valleys have been filled with drift, no narrow gorges choked by moraines to form natural impounding reservoirs, and no long *kames* to retard the surface-water and enlarge swamp areas in the more elevated regions. While the Potomac was uninterruptedly scouring its way down to bed rock, many of the northern rivers were crowded out of their valleys and obliged to continue their work of transportation at a much higher level and to create new channels. Now, if such an excellent study as Mr. Babb has made of the Potomac could be repeated in a more northern section of the country, perhaps in relation to the Connecticut, the Merrimac or the Penobscot, having like regard to the geology, total area, difference in elevation between source and outlet, rainfall, volume of discharge and quantity of sediment, the comparison would be very interesting. Although somewhat paradoxical, it might appear that the deposition of a vast amount of loose material in a well-eroded valley and upon its watershed, actually retarded the removal of the material toward the sea through its effect in equalizing the flow and thereby lessening the velocity and carrying capacity of the water. In determining the amount of sediment from the drift area, some other method than that described might be introduced, as in a lake or swamp district the color of the water would be affected by vegetable substances hardly to be called sediment as that word is generally understood.

The importance of such a paper as this relating to the Potomac cannot be too highly rated. Questions bearing upon water supply and water power can only be satisfactorily answered after such studies have been made, and greater attention is being yearly given to both of these subjects.

Mr. F. H. NEWELL.—The hydrographic measurements made upon the Potomac River form a part of the general study, carried on by members of the Geological Survey, of the forces modifying the earth's surface and of the resultant phenomena. In the particular matter in hand, viz., the distribution of water in rivers and lakes, and the variation in amount from month to month and year to year, there has been a great lack of data, and, as a consequence, the efforts of the Survey have been concentrated largely upon the acquisition of facts and in bringing together the comparatively fragmentary results of measurements made throughout the United States.

The observations made on the Potomac by Mr. Babb and the computations resulting from the use of the acquired data, begin to show, not only the character of the stream, but also furnish material for comparison with other rivers. One of the most striking facts is the extent of the fluctuations in quantity discharged, the average for one year, 1889, being over twice the amount for the preceding year, and nearly three times the quantity discharged the year before that. This great variation by any one river is a matter not commonly recognized by the public, and, when attention is called to it, many engineers are apt to attribute it to local or temporary causes. As a matter of fact, however, the study of the rivers of the United States shows that all are subject to what may be called non-periodic oscillations, viz., to a progressive increase in amount of water discharged through a period of years, it may be two, five, ten, or any number, followed by a decrease of greater or less rapidity and extending through several years.

The term non-periodic is applied to these general oscillations of indefinite length, to distinguish them from the periodic oscillations due to the annual range in precipitation, temperature, wind movement, and other climatic factors. This comparatively regular variation in river flow throughout the year can be ascertained by a few years' observation, and the characteristics of the river as to the usual time of floods and droughts, their duration and intensity, can be determined. By setting aside the effects due merely to these annual changes it becomes possible to study non-periodic changes and to make comparisons of one river system with another.

Such measurements as these on the Potomac and other rivers of the United States, while they have not been continued for a number of years sufficient to give full information, yet furnish indications of the relative importance of some of the factors modifying river flow. The discharge of a stream is the resultant, not only of the quantity of rainfall, but of the rate of precipitation, the temperature, wind movement, condition of the ground, whether wet or dry, its inclination, the pervious character of the soil, and many other conditions. Each modifies the other to such an extent that, in a large drainage basin at least, the measurement of any one, for example, rainfall, is of very small value in determining the quantity of river flow, for, as shown by hydrographic work in the West, a comparatively large annual precipitation may or may not be accompanied by an increased river flow.

Different combinations of the more important factors, viz., precipitation, temperature and wind movement, may have a cumulative effect in one direction or the other in increasing or diminishing the river flow to a remarkable degree. On the one hand, heavy precipitation, prevailing low temperature above freezing, and slight wind movement may give a great run-off, and in another year a slightly less precipitation, high temperature and considerable wind movement may result in nearly

dry channels in the streams. The fact to be dwelt upon is that small, almost insignificant, variations in each of the climatic factors produce results whose importance as regards river flow is immeasurably beyond the changes wrought by man on the earth's surface.

Two misconceptions, viz., on the one hand, the failure to appreciate the results of climatic oscillations, and on the other, the tendency to exaggerate the importance of changes due to the advance of civilization, have resulted disastrously in many ways. As one example, may be cited the attempts made to introduce agriculture upon the western edge of the Great Plains, where, from lack of streams, irrigation is impossible. It so happened that about the years 1884, 1885 and 1886 there was an unusual amount of moisture in many localities where summer droughts usually prevail. The conclusion was immediately reached that the rainfall on the plains was increasing and with the advance of settlement a permanent improvement of climate had resulted. Hundreds and even thousands of families were induced to settle upon these dry plains by the statement, in many cases stamped with official approval, that the rainfall was increasing. Hardly was settlement well under way before the climatic oscillations took the downward turn, and after three, four, and even five years of struggle, the farmers, who had hardly been able to recover the seed from the ground, were literally starved out and many were compelled to solicit aid in order to leave the country. Experience shows that crops can probably be raised again as they have been in past years, but while waiting for the fortunate year an agricultural population will die of famine.

Other examples of climatic oscillation may be cited in the case of important streams of Colorado and other States which declined from about 1885 to 1888, in the latter year discharging only one-fourth or even one-fifth that of former years. These have been held as examples of the evil results of destroying the forests, but, unfortunately for the theory, a continuation of measurements and a more general study show that this decline is not continuous nor confined to localities where the mountains have been laid bare. Neither does a close examination of the data furnish support for the popular belief that the spring floods are as a rule higher or more destructive, or the summer droughts longer or more severe, the oldest inhabitant to the contrary.

To sum up the results of hydrographic measurements so far as they have been carried on, the data point toward the facts that variations in river flow are exceedingly irregular, and are so greatly modified by comparatively small changes in climatic factors that in a large drainage basin at least, measurements of any one, as of rainfall, as ordinarily conducted, furnish little information as to the quantity discharged. Over adjacent drainage areas there is a general similarity in variations of river flow, but the attempt to extend comparisons over a considerable fraction of the country is often fruitless.

CYRUS C. BABB, Jun. Am. Soc. C. E.—The writer has in course of preparation a similar paper to the above on the Connecticut River Basin, based upon the measurements of Gen. Ellis, from February, 1871, to December, 1877, on gauge-height observations maintained since that date, and upon the rainfall records of the Weather Bureau, and he is in hopes of being able to present some interesting comparisons between the two basins.

Of late years engineers are coming to recognize the futility of computations of river-flow based on empirical formulas. The present writer, fully recognizing this fact and believing that computations should only be based upon actual measurements, wishes, however, to present a curious fact found amongst his figures. In his velocity measurements upon the Potomac River, the integration system was at first used, but afterwards discontinued for velocity measurements at separate points in the vertical cross-section. From these measurements vertical velocity curves were constructed. The mean velocity for each vertical section was then computed with its corresponding depth, and what he desires to call especial attention to, the ratio of the depth of the mean velocity to the total depth was found, as shown in the following table, which gives for each date this ratio for every ten feet across the river.

	30	35	40	50	60	70	80	90	100	110	120	130	140	Mean.
June 13th	.60	.61	.55	.53	.62	.54	.55	....	....	....	....	....	....	.571
December 15th	.57	....	.58	.54	.53	.58	.58	....	.62	.55	.53	.51	.66	.568
" 22d	.60	....	....	.60	.64	.68	.64	.61	.62	.63	.57	.62	....	.621
" 29th	.56	....	.60	.56	....	.56	....	.56	.58	.61	.53	.60	.55	.571
Mean	.58	.61	.58	.56	.60	.59	.59	.59	.61	.60	.54	.58	.60	.583

The Mississippi River Commission found from their work a ratio of 0.622. Mr. E. E. Haskell, of the United States Coast and Geodetic Survey, stated to the writer that from some measurements that he had occasion to make he considered the ratio of 0.622 too high.

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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EMERGENCIES ON RAILROADS.

By CHANNING M. BOLTON, M. Am. Soc. C. E.

WITH DISCUSSION.

A great deal has been published on the subjects of construction and maintenance of way of railroads, much of which seems to the author very useful and a great deal of which is of little or no value to the average engineer. Having, however, had much experience in the class of work upon which he proposes to write, he has concluded to venture, hoping that what he has to say may be found of use to some members of the engineering profession who may be called on "in an emergency."

What to do in an Emergency.—Of course, on "our roads" we should have no emergencies, but should be fully prepared for any contingencies that may arise. No doubt, we would be so prepared, but our Finance Committees, Boards of Directors, Presidents, General Managers, and Treasurers are sometimes not able to see, or if able to see, not willing to allow "a stitch in time." Taking into consideration, however, all these contingencies and allowing to those who are always fully prepared the privilege of not reading this article, a few practical hints will be given by the writer, who regrets to have to acknowledge that

he has frequently had to learn by experience to act very promptly, when at the same time he was very poorly equipped for his work.

Removal of a Wreck on Ordinary Cut or Fill when the Track is Obstructed.—The first object is to open the road to traffic. The next is to decide which cars and materials are worth saving and to work with this end in view, cutting up such wreckage as may seem worthless, so that it can be the more easily removed. Then place "dead men," or select trees or other stable objects if they can be found in proper position from the wreck and sufficiently rigid, to be used as fixed points to which to attach snatch blocks. These are to be rigged with 2-inch manilla ropes, or larger if necessary, which are to be attached to a locomotive and also to such parts of the wreck as it may seem best to draw out. A wire rope, such as is described in the accompanying list of necessary tools, should be used in taking hold of the wreckage, being hitched on to the manilla rope for this purpose.

Removal of a Wreck Occasioned by the Fall of a Bridge.—1st. Decide what is worth trying to save and remove it as soon as possible.

2d. Measure at once the height necessary for a trestle; and if there are timbers on hand that will answer for such purposes, order them up; if not, make arrangements to have them cut and delivered promptly, ordering about 50 per cent. more of heavy timbers and about double as much bracing plank, 2 and 3 inches thick, from 16 to 30 feet long—as the net estimate calls for. The surplus will be needed for scaffolding, etc. If logs of from 12 to 18 inches diameter can be obtained, they will answer better than square timber for posts and be much more economical. If practicable, have half of the material delivered on each end of the wreck.

3d. If in a river or other water, where boats can be used to advantage, start a carpenter force to work immediately at making three boats about 8 x 26 feet, as per sketch (Fig. 2, page 47), and one rowboat about $2\frac{1}{2}$ x 18 feet, if you have no such boats on hand. One large boat is to be used in transporting men and material, the other two as described below.

4th. Order a telegraph operator to the spot at once, and open an office near your work.

5th. Estimate what forces are needed, according to your best judgment, and order them to be hurried to the work.

6th. If the case is one where you will be detained several days, divide the force up into day and night squads, and order the squads

not at work to keep out of the way and to report promptly at a designated hour.

7th. The distribution of forces should be, a squad of about twelve carpenters at each end of the bridge for framing and raising; a squad of about twenty laborers with an intelligent foreman, to attend each carpenter force to supply it with materials, and to furnish additional assistance to handle heavy materials.

8th. Be sure to have plenty of 6 and 8-inch spikes, both cut and wrought; drift bolts about 22 x $\frac{3}{4}$ inches or $\frac{5}{8}$-inch diameter, and ordinary screw bolts $\frac{5}{8}$ and $\frac{3}{4}$ inches diameter and 24 inches long, with nuts and cast washers.

9th. A good force should be started promptly on the work, chipping with cold chisels and loosening bolts where they are likely to hang, so that the parts can be pulled out in detail.

10th. Stretch a rope, if possible, across the stream, to be used as a guide to the boats in taking men and materials backward and forward.

11th. Under certain circumstances the wreck can be handled by an engine on the main track, with snatch blocks as heretofore described; but usually it is best to build a temporary track down the hillside to the bottoms, and then turn this track on the bottom, down or up stream as may be found best; the engine is then in position to do the most effective work, either working directly with ropes attached to parts of the wreck, or by passing ropes through snatch blocks as described and then to the wreck.

12th. When night work is to be done, arrange during the day for proper lights, and place them in charge of an intelligent man detailed for this purpose. The author has found the Wells' lights very satisfactory; the ordinary gasoline lamp, such as is used at fruit stands, etc., is also very useful. This is in case no electric plant, such as is hereafter described, is to be had.

13th. After the wreck is removed, the next step is to raise the trestle. Place lever arm timbers, 12 x 12 inches, about 30 feet long, with sheaves set in them at about 2 feet from the ends, and fasten double block with 1-inch rope falls about 1 foot from the ends, as per Fig. 1, page 46, so that the outer ends of the timbers will be directly over the position in which it is desired to set the post. Support the lever arm timbers at a point near the center at the proper distance from the sheave on the abutment or next trestle; block it up so as to raise the sheave ends of

the timbers about 2 feet above the top of the caps; then secure the inner ends of the lever timbers so that they will withstand the leverage when lifting timbers at the outer end. Having fastened guide planks to two of the large boats, as shown in Fig. 2, page 47 (which planks should project from one side of the boats, so as to make the distance from their ends to the far side of the boat the distance desired between the trestle bents), place the boat with the current, allowing the ends of the guide planks to rest against the abutment; this will place the outer side of the boat in the proper position for the first bent. Then float the trestle post about under the position in which it is to be set, having a chain or rope sling placed around it a short distance above its center of gravity, and raise it with the fall rope until the lower end is above the surface of the water; then loosen the fall rope, dropping the post in the proper position on the bottom, using the outer side of the boat for a guide.

If the bottom is soft or loose, churn the timber down with cant hooks; if there is an irregular rock bottom, select interstices or low places between projections on the bottom for the posts to rest on, though they may not be exactly in line. After the first post is set on the bottom, run a 3-inch plank out from the abutment, spike the outer end to the post, plumb the post, and fasten the other end of the projecting plank to the abutment. Then raise the other three legs of the bent in like manner, raising the two center posts first, righting them up, and bracing them laterally to each other as soon as raised. After having raised the two center posts of the first bent, spike a guide plank to one side, of a length sufficient to take in all four posts when raised, being careful to have the upper edge placed perfectly horizontal and about 24 inches below the position that the lower chord of the permanent bridge will occupy. Place two lines of horizontal braces of about 3 x 12-inch plank about 4 feet below the top of the caps, on the outsides of the center posts and parallel to the axis of the bridge; these will act both as bracing and scaffolding for the carpenters (see Fig. 1, page 46). Then saw off the tops of the posts flush with the top of the guide plank; place the cap (which should be 22 feet long in order to use it as false work in raising the permanent bridge) on top of the posts and drift-bolt it to them with bolts of five-eighths or three-fourths inch diameter, and 22 inches long, and block up on top of the caps to the proper height for the stringers.

When the first bent is raised and properly braced, move the "guide boat" and lever arms forward, using the first bent in the same manner as the abutment was used in the start, and so on until the forces working in the same way from the other end of the span are met.

14th. Block up on the caps to the proper height, and put stringers, cross-ties and rails in place.

15th. The above description is for cases where the height is not too great for a single-deck trestle. Where additional height is required, while part of the carpenter force is raising the first deck, others should frame the second deck, having determined its height by deciding to what height the first deck will be built.

16th. In cases where there is little or no water, the above-described method may be used for clearing the wreck, but raising the trestle is simplified by framing bents to the proper height and raising them by use of ropes attached to a locomotive.

An engineer should take charge of the work himself, and it should be understood that there is but one man in charge. He should see that his forces never have to wait for materials after the work has been started. These notes are suggestive in case of an emergency and when an engineer is poorly prepared to meet it. A memorandum is given below of such things as every company should have at a convenient position on its line, and which, in the above-described contingencies, will make the task much easier.

First.—One or more good wrecking cars.

Second.—A car arranged with a complete electric plant sufficient to run ten arc lights, a part of the car being used for the storage of small tools.

Third.—4 Wells' lights; one-half dozen gasoline lamps, such as are used on fruit stands, etc.

2 iron snatch blocks for 3½-inch diameter rope.

2 iron snatch blocks for 2-inch diameter rope.

2 pairs iron (two single and two double) blocks, 2-inch diameter rope.

2 pairs iron (two single and two double) blocks, ¾-inch diameter rope.

6 pairs iron (six single and six double) blocks, 1-inch diameter rope.

4 large 5½-foot cross-cut saws.

4 small 4-foot cross-cut saws.

6 pinch bars, octagon steel, ¾-inch diameter	One end chisel pointed, the other drawn to a sharp point.
6 pinch bars, octagon steel, ¾-inch diameter	

12 track claw bars.
12 crow-bars.
12 spiking mauls.
12 cold chisels (cleavers).
12 cold chisels (hand).
4 boring machines, assorted bits.
6 ship augers, ⅜-inch.
6 ship augers, ¾-inch.
6 ship augers, 1-inch.
4 ship augers, 1½-inch.
6 adzes.
12 pole axes and helves.
12 hand hammers, 4 pounds.
4 hand hammers, 6 "
4 hand hammers, 8 "
4 hand hammers, 12 "
2 grindstones.
1 dozen water buckets.
12 kegs 6-inch cut spikes.
12 kegs 8-inch cut spikes.
12 kegs 8-inch wrought spikes.
12 kegs 6-inch wrought spikes.
5 kegs 12-penny nails.
200 ⅝-inch screw-bolts, 18 and 24 inches long.
200 ¾-inch screw-bolts, 18 and 24 " "
400 cast washers, for ⅝-inch bolts.
400 cast washers, for ¾-inch " .
100 drift bolts, ¾-inch diameter, 22 inches long.
12 1-inch manilla rope slings, from 4 to 6 feet long.
12 2-inch manilla rope slings, from 4 to 6 " "
6 pliable wire ropes, 1 inch diameter, 50 feet long, hook at one end, link at the other end.
2 pliable wire ropes, 1 inch diameter, 100 feet long, hook at one end, link at the other end.

2 pliable wire ropes, 1 inch diameter, 30 feet long, hook at one end, link at the other end.
12 chain shackles, see Fig. 4, page 47.
2 dozen shovels.
1 dozen picks.
12 boat poles, with hooks, as per Fig. 3, page 47.
24 "log dogs," as per Fig. 5, page 47, for fastening ropes, when drifting timber.
1 coil 3½-inch manilla rope.
4 fall ropes, 1 inch diameter, 200 feet long.
1 extra coil, 1 inch diameter.
1 extra coil, ¾-inch "
1 bale of oakum.
1 barrel pitch.
1 dozen wrenches of various sizes.
½ dozen monkey wrenches.
4 chains, ½-inch iron, 6 feet long, a ring on one end of sufficient size for chain to pass through, and a hook on the other end.
4 chains, ½-inch iron, 8 feet long, a ring on one end of sufficient size for chain to pass through and a hook on the other end.
4 chains, ¾-inch iron, 10 feet long, hook on one end and link on the other end.
4 chains, 1-inch iron, 10 feet long, hook on one end and link on the other end.
3 boats, as per Fig. 2, page 47.
1 rowboat, 2½ feet wide by 18 feet long.

Fourth.—It is taken for granted that the carpenters will have the necessary hand tools, such as are usually found in a mechanic's chest.

Fifth.—Bill of timber which should be kept always on hand:

50 stringers, 7 x 14 inches, 25 to 30 feet long.
30 caps, 12 x 12 inches, 22 feet long.
125 poles, 12 to 18 inches diameter, 25 to 40 feet long.
200 braces, 3 x 12 inches, 16 to 30 feet long.
200 braces, 2 x 12 inches, 16 to 30 feet long.

Sixth.—As soon as a train has passed over, have the tools checked up and put in their proper places, replacing such as have been lost or destroyed, and have all put in order for another "emergency."

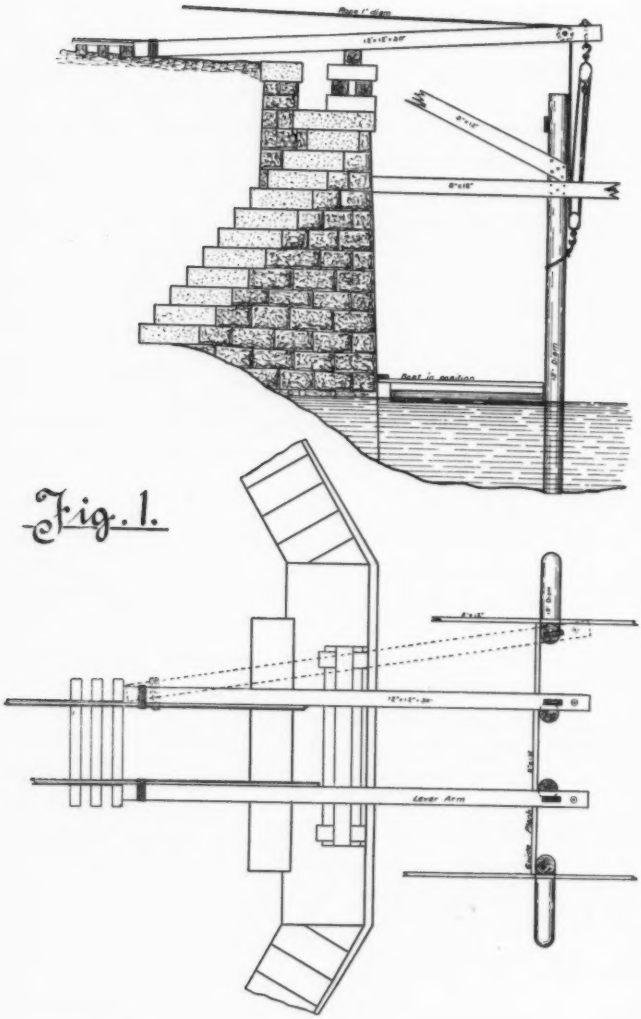


Fig. 1.

Fig. 2.

Fig. 3.

Fig. 4.

Fig. 5.

DISCUSSION.

W. G. CURTIS, M. Am. Soc. C. E.—Webster defines an emergency as "any event or combination of circumstances which calls for immediate action or remedy; pressing necessity; exigency."

While, by reading and discussion, a person may prepare for emergencies of almost every imaginable kind, those which relate to the interruption of railway traffic, through damage caused by storms, fire, flood, failure of mechanical structures, train collisions, derailments, etc., require remedies that can best be learned through practical experience. Aside from the skill growing out of such experience, the successful man must be full of resource, able to plan quickly and correctly and execute rapidly; being quick to take advantage of every favoring circumstance, and unerring in his judgment in the selection of his foremen, who often times must be picked out from a large force of men hastily assembled. Only the most general principles can be laid down for the guidance of the novice. I have sometimes directed and many times witnessed the removal of minor train wrecks, by experienced railroad men with improvised appliances, in a shorter time than less skillful men could accomplish similar work with a "wrecking car" and a first-class equipment of tools.

In a great emergency requiring a large force of men, the commissariat is the first thing to be provided for. Under skillful leadership, men can be directed to accomplish almost everything, provided, always, that their food supply is adequate and promptly forthcoming when required. Whatever else is done, the cooks must be on duty at all times, and those who keep up the supply of hot coffee for the night force are particularly indispensable.

The paramount consideration is to prepare the road for resumption of traffic. No time is allowable for refinements of any kind. A good theoretical, as well as practical, knowledge of the strength of materials is desirable in building temporary structures supporting the track, and an engineer ought to be, and generally is, the best man to take such work in charge, but it is well to remember the moral of the story about the man who reported a bridge completed and ready for an army to cross a river, but he "didn't know whether the engineers had the 'picturs' done or not."

SAMUEL M. ROWE, M. Am. Soc. C. E.—The practice of many railroad companies being to dispense with the services of the experienced and competent civil engineer largely; and where he is retained, confining his jurisdiction and authority to the more general questions of plans, estimates and general policy, the matters treated of in Mr. Bolton's paper seem to be but remotely connected with the profession, or those questions that may be discussed by the Society with any great

degree of profit. That this is as it should be, is not now the question. It was said by a professional engineer, here, in our city, that the popular conception of an engineer was, "a man with blue overalls and an oil can, with most of the contents of the can distributed over his clothes." But it seems as if the members of the profession could hardly command even this degree of importance.

Too often, even the wisest and most practical suggestions are rejected because they come from that source, and the work is left to men who have the dash to accomplish it by main strength and awkwardness.

But the especial value of the paper comes from the fact mentioned in the second paragraph, *i. e.*, the writer's experience, and may be well supplemented by the experience of others.

It often comes into the province of the Resident Engineer to take such precautions as will meet the "stitch in time" in a measure, by making estimates of material and tools needed, and also furnishing such a record of structures, as will enable the operating department to act promptly in case of accident. I have in mind a case where the Division Superintendent was furnished with a complete list and an elaborate record of bridge structures, including concise description, and showing length, number and height of bents or piers, and all the data necessary to determine at a glance, just what would be required to rebuild, in case of its collapse.

The list of tools given is very good. I would add what every railroad should have. That is a steam pile-driver, with extension equal to the ordinary panel length of the pile bridges. With this, all pile bents could be renewed, trestle or false-work could be raised, and any but the heaviest lifting could be done. I have seen this machine used with excellent results, as a wrecker, saving any amount of manual labor.

C. Palmer, M. Am. Soc. C. E.—It has been the writer's fortune to have enjoyed (or otherwise) an experience similar to Mr. Bolton's, and he can truly say that the situations sometimes encountered try men's souls. On the very day the advance copy of his paper was received, a deck bridge consisting of two combination spans of 141½ feet each, with trestle approaches 208 and 80 feet respectively, was found on fire. The track is 43 feet above the water, which is 4 to 8 feet deep, on a rocky bottom extending about half way across the stream. The trestle approaches are 32 feet high. The current of the stream is 5 or 6 miles per hour, and rapids occur just above and below the bridge. It is a stream on which extensive logging operations are carried on, and active driving is now in progress. Thirty-one trains each way, or sixty-two in all, cross the bridge every twenty-four hours. Fortunately, the short intervals between trains led to the discovery of the fire before serious damage was done, but with twenty or thirty minutes more headway it would have been beyond control. Had the bridge been destroyed, how could a temporary crossing have been constructed with the least pos-

sible delay to traffic, and what would have been the least time in which it could have been done? The plan illustrated by Mr. Bolton would not have worked. Even piles, were there sufficient earth to hold them (and there is across half the river), would be swept away by the running logs faster than they could be driven. The writer leaves the problem unsolved for the present, though advised to put on his thinking cap.

The circumstance is dwelt on at considerable length because it illustrates that no cut and dried plan can be devised to meet emergencies of this character. There are absolutely no parallel cases. I have never had a case to which Mr. Bolton's plan would have applied. The engineer must simply tax his ingenuity for a solution of each problem as it presents itself. Perhaps a brief narration of a few experiences with this class of emergencies may have some interest, and an occasional suggestion might happen to stray out of it that would be of service in other cases.

A riveted lattice of 117-foot span was partially wrecked by a load of logs. The front end of a log was hanging so far over the car body that one-half of its diameter struck the end brace of the bridge. The brace was bent out and the log slipped by it, raking past two or three braces till it finally became entangled with the members and stopped the train. Two main braces and two counters were broken out, and other members more or less damaged. The logs were let through into the river and the train passed on. The accident occurred in the night. One other train had passed over the bridge after the accident, before the writer arrived. Had he not known that fact, he would have pronounced the bridge unsafe; but what had been done, might be, and trains were allowed to pass. Fortunately, the water was only 3 feet deep and the bottom was hard, and there was timber near by. Temporary posts were at once set under the damaged truss and remained until the bridge was repaired.

Another stream is crossed by four spans of wooden Howe truss 121 feet each. The forward truck of a freight car in the middle of a train was derailed 1 000 feet from the bridge—was dragged unnoticed (it was after dark in the evening) over the frozen ground and struck the end brace, breaking it completely off, resulting necessarily in breaking the chord at the first panel point, which dropped down about 2 feet. Two cars were derailed and jammed into the bridge, and the train was stopped. The writer arrived on the ground at daylight next morning with men and material. The wrecked cars had been removed. Screw-jacks were set under the broken chord and by ten o'clock it was in position, though not sufficently supported to carry trains. In this case there was another road parallel with ours and connecting tracks within a short distance on both sides of the stream. Arrangements had been made for crossing their bridge, and it was deemed advisable to take advantage of that circumstance, and, instead of building a temporary

abutment under the broken chord, to put in temporary work for supporting the bridge during repairs. This was done and traffic resumed the following morning.

Another case was that of a wooden Howe truss deck span, 130 feet long. The chord timber had not been properly selected and was inferior in quality, and began to exhibit weakness by opening of the chord joints within a year after its erection. The chord was composed of four members 7¼ x 14 inches, and there was one joint in each panel. Finally, while making some slight repairs on the approaches, the foreman of a bridge crew discovered a broken member in the center panel. The joint member in this panel was outside and the broken one was next to it. There remained but 14 x 15 inches of timber in the chord, the other members of which were shearing upon one another, and all the joints in the adjacent panels exhibited the distortion, which had also reached to the opposite chord. Fortunately, again, standing timber was adjacent, and before another train was due the foreman had a post under the break, and two more soon after. The bridge had settled to that side, and was righted up after the posts had settled to a solid bearing. It was finally jacked into shape, its camber restored and permanently repaired.

The next case relates to track. A sand embankment 25 feet high carries two tracks, and along the foot of the bank runs a track that forms the main artery between the yard and the main line, all being in the yard. Trains or switch engines are passing over the low track every few minutes, and over the embankment tracks only a little less frequently. There was originally a timber retaining-wall confining the foot of the slope to make room for the low track. It became necessary to raise the grade of the embankment, necessitating a higher retaining-wall. The work was done during winter. The timber was taken away while the frost held the bank, and a dry limestone wall built in its place; but, in consequence of change in height or center, it became necessary to build the wall twice as high as was first contemplated, and it proved to be too light. When the frost went out in the spring, the wall began to fail. It was a serious matter to lose the use of either of the tracks, and the writer was called upon to hold the wall. For a considerable part of its length there was not room between the wall and the low track to set braces, though that was done where it could be. The plan adopted for the rest of it is shown in Figure 1. To get the long rods in, it was necessary that they should be driven through from the wall side of the bank. But trains or engines were passing there every few minutes, and it would take an hour or two to drive a rod through. Gas pipe, 1½ inches diameter, was cut into 5-foot lengths and threaded at both ends. One piece was put through the wall and driven in; another piece was coupled on to it and followed, and so on till the end of the first piece appeared on the opposite side

of the bank. A solid head to receive the blows of the ram was fitted to the end of the pipe, being screwed on and removed from the lengths successively. When the pipe was through the bank, the rod, being in position on pole tripods, was screwed into a coupling on its end, and, attaching a line to the head on its other end, four or five men pulled the pipe out and the rod in. The rods were spaced 12 feet apart and longitudinal timbers were run behind the stay posts along the wall as shown. Each operation occupied but a few minutes, and trains were not interrupted.

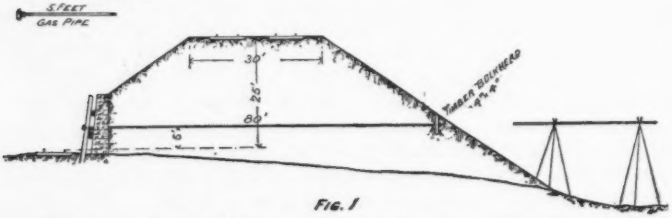


FIG. 1

These are characteristic cases. It is the unexpected that happens and no one can be prepared at all points. The complete destruction of a bridge contemplated in the plan of operations, illustrated by Mr. Bolton, must be treated with some patience. Railroad companies are usually supplied with tools and material for ordinary cases. Undoubtedly, it is well to anticipate extraordinary cases also; but with a well-equipped shop and a saw mill or forest accessible, one can always find a way. Companies are disposed to deal liberally in matters of that kind and to tolerate expense, if results are promptly secured.

The list of supplies given by Mr. Bolton is certainly generous. The writer would like to draw on his stock of tools when he gets his list filled.

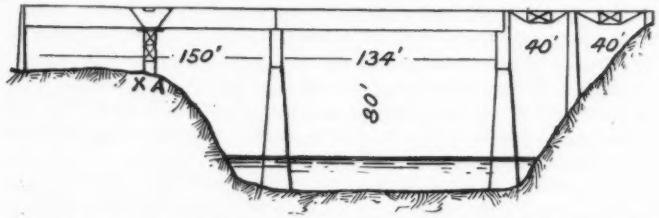
Bolton W. De Courcy, M. Am. Soc. C. E.—An eminent officer in the British Navy told me, when I was a youngster, that he attributed his success in life to a habit he acquired, when young, of building an emergency which is a perplexing contingency, as a *Château d'Espagne*, and devising, as he lay awake at night, what would be the best way to meet it. I have followed his recommendation as well as I could, and several times this habit has saved my life.

One of these times was when I had charge of a road as supervisor and bridge engineer, and used a three-wheeled velocipede a great deal. There were many rock cuts on the road, and several were so narrow that we had to remove after winter frosts, rock, that, having been forced out, was in danger of scratching the coaches; several of these cuts had very sharp curvature also. One day, as I was passing through one of these cuts, I put the question to myself, what should I do, should I meet an engine? I decided the only way out would be to jump, and

at the same time over-set the velocipede to the right, and I immediately made trial to see if I could carry out my answer to the question, and I succeeded without hurting myself.

Sometime afterwards, I was going to inspect a bridge, and had the foreman of the bridge gang riding behind me, a man seventy-five years of age, but active as a cat. There was a station 2 miles from the bridge, at which we inquired whether the local freight, due in half an hour, ever ran in ahead of time; it was not a telegraph office. The answer was, never; so, being in haste, we started ahead and met the train making a run so as to make a running switch at the station in order to make time. I threw myself to the right and jumped at the same time, catching the axle of the small wheel and throwing my back against the rock wall of the cut we were in (about 18 feet deep and on a 10-degree curve). It was done so quickly, the engineer thought he had run over us and so reported at the station. Not even the velocipede was injured; we ran on to the bridge, made the inspection and returned, catching the freight at the station, and relieved the minds of the conductor and engineer who were almost frantic.

When I was appointed on this road, it had just passed through a period of economical administration, and had been run without any



roadmaster; their number of wrecks averaged two a week, and I was kept extremely busy on track. On my inspection of the bridge mentioned above, I found a deck Howe truss, composed of two spans, one 150 feet, the other 134, and two trussed stringers 40-foot span each. It was an old bridge, and I was informed had been framed at the car shop by the master carpenter; the height of trusses was 11½ feet. There was no difference in the length of the run of the panels of the top and bottom chords, an attempt having been made to frame the camber in the braces and without avail. The shorter span was somewhat sagged; the longer had sagged so much that they had put in a single trestle at a place at one side of the center panel. This had cracked the lower chord at each side and I found nearly every angle block crushed.

My objective point just then was the telegraph office, and I immediately wired that no train should or could pass over the bridge until repaired. I received an answer that all trains must run and that I should be held responsible for the lives of the traveling public. I

started to write my resignation, when the station agent reminded me of its being Saturday, and after the next train there was none other until 5 P.M. Sunday. I then altered my intention and wired for men, tools and timbers. The shape of the gorge passed was as sketched. At *X* was the temporary bent or trestle, and on examination I found that there was an odd number of panels in the span. I framed an extra bent, putting it at a place marked *A*. I then placed some long corbels under the chords at the top, so eliminating the panel in which the chords were cracked. I then removed the rods, a panel at a time; took out the braces and placed new angle blocks, and reversed the braces and counters for half the distance on each side of the temporary pier I had made of the two trestles, thus making two spans; and I had the work all finished ten minutes before the passenger train was due. While on this road there were two spans of 121 feet each or thereabouts, of through housed-in Howe trusses burned. The depth of water was 7 to 9 feet; we put in the temporary bridge in less than a day. We framed and put together our trestles, on the ends of the spans, and lowered them to place all together, sliding them down two slanting timbers which reached the proper position at the bottom and then plumbed them with hawsers. We found this more expeditious than building them in place.

I have often thought that a trestle with shoe and movable cap, such as is used in the Austrian military bridge, would be of great service in emergencies arising on railroads, as it is a great help to get a temporary bridge across with as much expedition as possible. Mr. Bolton's list of tools and materials is good, and all things that can be foreseen to be needed in emergencies should be held in depot at convenient places for such use.

CHANNING M. BOLTON, M. Am. Soc. C. E.—Mr. W. G. Curtis has made a good point—"the cook" is certainly a very important adjunct and is one of the points that I have always carefully looked after in an "emergency." It does not seem to me that my suggestions on "emergencies" are so pictorial, that it would take longer to build the bridge than draw the "pictures"; it is only a memorandum of work that I have actually done with great rapidity and success. The speed with which I have accomplished such work is one bent of trestle in two hours with each force, or working at each end one bent per hour, or completing a span of 150 feet in twelve hours, and allowing trains to pass at the end of that time.

The addition of a pile-driver, as suggested by Mr. Rowe, is good, but I have never had a case where I could take the time to use it in an "emergency." I have always been able to get the timbers to stand without it, and afterwards used pile-drivers for more permanent work.

In the case cited by Mr. Palmer of a current of 5 or 6 miles per hour, 4 to 8 feet of water, with rock bottom, and track 43 feet above the water, etc., I see no difficulty in the way of using the plan sug-

gested by me, as I have had a very similar case where the water was about as swift and 6 to 10 feet deep, with the track 60 feet above the water, and a considerable amount of drift, probably, however, not as much as in the case cited by Mr. Palmer. In this case there were four spans, only two of which were knocked down. I kept men diverting the drift until the trestle was finished, and then built a floating boom extending from the shore around the newly erected trestles, which stood perfectly for four to five months until a new span of iron bridge could be constructed and erected; in the meantime the water rose to a depth of about 25 feet. I would like to know the plan of the original false work for raising the bridge mentioned by Mr. Palmer. I think that if he will look into it, he will find it was very similar to the one proposed by me. I am afraid from the last remarks of Mr. Palmer that he is disposed to approve of the policy of companies, such as Mr. Rowe mentions, of employing as engineer "a man with blue overalls and an oil can," etc., and not one who looks ahead and works with system, even though it be in an emergency.

The list of tools is not what we all have, but I think, in fact know, is one which all large systems should have; smaller roads "cutting the garment to suit the cloth." Frequently, for the lack of proper tools and materials, the delay to trains for a single wreck will cost more than the whole list of tools, etc., mentioned by me. I am aware that no fixed rules can be made for all emergencies, or there would be no use for engineers, and this fact might be stated of the profession generally. The engineer should be a man able to plan and execute large work, but in "emergencies" should also be able to think and act quickly, making the most of his surroundings, economizing judiciously, both time and money.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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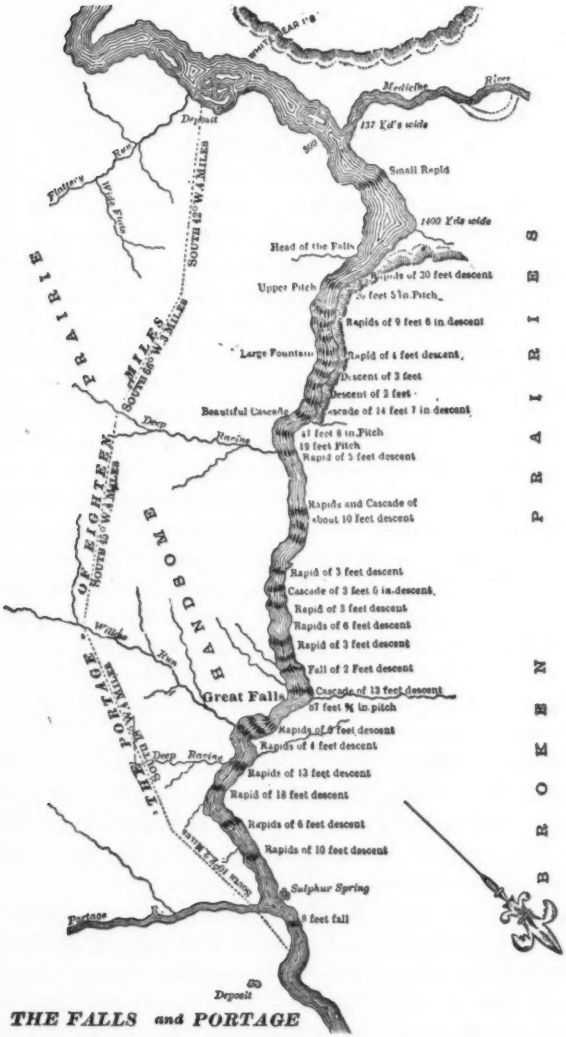
BLACK EAGLE FALLS DAM, GREAT FALLS, MONTANA.

By MAURICE S. PARKER, M. Am. Soc. C. E.

WITH DISCUSSION.

The falls of the Missouri River were first made known to the geographical world in the year 1805, by the explorers Lewis and Clarke on their expedition of exploration to the headwaters of the Missouri. The map of the river at the falls, which accompanies their report (see page 57) is remarkably accurate and is a monument to the faithful work of these early explorers. They did not indicate the actual fall of the river on their map, giving only the height of rapids and falls. The total fall of the river for a distance of 10 miles from the first rapid to the foot of the fall known as the "Great Falls" (which has a sheer drop of 75.5 feet) is 412.5 feet.


Medicine River
137 Yd's wide
Small Rapid
1400 Yds wide
Head of the Falls
Upper Pitch
Rapids of 30 feet descent
Rapids of 9 feet 6 in descent
Rapid of 4 feet descent
Descent of 3 feet
Descent of 2 feet
Cascade of 14 feet 7 in descent
Rapid of 3 feet descent
Rapids and Cascade of about 10 feet descent
Rapid of 3 feet descent
Cascade of 3 feet 6 in. descent
Rapid of 3 feet descent
Rapids of 6 feet descent
Rapid of 3 feet descent
Fall of 2 Feet descent
Great Falls
Cascade of 13 feet descent
87 feet ½ in pitch
Rapids of 8 feet descent
Rapids of 4 feet descent
Rapids of 12 feet descent
Rapid of 18 feet descent
Rapids of 6 feet descent
Rapids of 10 feet descent
Sulphur Spring
8 feet fall
Deposit
THE FALLS and PORTAGE
PRAIRIES
BROMEN
HANDSOME
PRAIRIE OF EIGHTEEN MILES

THIS IS AN EXACT FAC-SIMILE OF THE MAP MADE BY LEWIS & CLARK, WHO EXPLORED THE UPPER MISSOURI RIVER UNDER INSTRUCTIONS FROM PRESIDENT THOMAS JEFFERSON IN 1804 AND 1805.

Nearly all of this 412 feet is available for water-power. The minimum flow of the river is very nearly 4 000 cubic feet per second. By gauging the river at the site of dam at a very low stage of water in the latter part of September, 1890, the writer found a flow of 3 860 second-feet, and on September 15th, 1891, Mr. E. L. Nettleton, of the Agricultural Commission, found by gauging at the dam a flow of 3 885 second-feet. In January, 1891, the lowest stage of water of which there is any record was reached, the average flow for two days being 2 120 second-feet. The writer has since discovered that this low stage of water was caused by dams formed in the river by anchor ice on shallow bars and holding back the water. In the early part of December, 1891, the writer came down the Missouri in a boat from a point about 150 miles above Great Falls. The weather was cold, freezing at night, and several of these ice dams or gorges were encountered, which in places backed the water to the height of several feet. Retardation of flow from this cause was very noticeable during the construction of the dam at Black Eagle Falls. In early winter the water would fall during the night and begin rising in the morning, continuing until about 12 o'clock, when it would reach its full height and begin receding. These dams form regularly at night during moderately freezing weather only, and give way under the influence of the sun of the succeeding day. As soon as solid ice forms on the river these conditions disappear and the flow is comparatively steady.

When, in 1887, the Great Northern Railway was built through the town of Great Falls, the value of this water-power became apparent to its projectors, and steps were at once taken to utilize a part of it for commercial purposes by constructing a dam at the first fall, known as Black Eagle; the result is that what only a few years ago was wild prairie, is now the site of a thriving city of nearly 10 000 inhabitants, which bids fair to rival the well-known cities that have grown great through the influence of water-power.

J. T. Fanning, M. Am. Soc. C. E., the well-known hydraulic engineer, made an examination of the ground at Black Eagle Falls in the summer of 1887, and prepared plans for a dam to be constructed just above the falls, whereby 40 feet of head was to be obtained for power. Nothing, however, was done toward construction until the spring of 1890. Bids, for labor of construction only, were opened February 5th, 1890, the contract requiring the completion of the work on or before

December 1st of the same year. The estimate upon which bids were solicited was as follows:

	Quantities.	Range of Bids.
Excavation, solid rock	9 000 cubic yards.	$1 18 to $4 20
Excavation, loose rock	2 000 "	49 to 1 00
Masonry in gate chambers	3 194 "	5 85 to 20 00
Masonry in crib of dam	3 600 "	1 85 to 6 50
Stone filling in crib, dry	5 300 "	1 45 to 6 00
Rip-rap	150 "	1 10 to 5 00
Back filling earth	500 "	25 to 90
Placing timber in dam	1 800 M. feet B. M.	6 00 to 13 50
Framing gates	86 " "	14 00 to 40 00
Placing gate gearing	40 tons.	14 00 to 30 00

The highest bid is omitted from the above table, it being out of reason. The range of the bids received for labor alone was from $63 000 to $305 000. All material was to be furnished by the Water Power Company at the site of the dam; the stone, however, was to be quarried by the contractor and handled by him, the Water-Power Company to pay freight in case the stone could not be obtained near the site of the dam. The lowest bidder failing to furnish the required bond for the completion of the work, and the next lowest bid being about $75 000 above the lowest, it was decided to reject all bids. Mr. J. T. Fanning, Consulting Engineer, engaged the writer to take charge, for the Water-Power Company, of the construction of the dam and to do the work by day labor. Nearly all of the bidders were either engineers or employed engineers in making up their bids, and as nothing new was claimed for the design of the dam, and such a wide difference in the bids of the contractors existed, the writer infers that the cost of construction of dams in swift running water is largely a matter of conjecture to many engineers.

The foundation upon which the dam is constructed is a close grained, hard, red sand rock, strongly impregnated with iron. This lies in vertical strata varying from 1 to 4 feet in thickness, the seams running at an angle of about 45 degrees to the direction of the current of the stream. Below this layer of sand rock, which is about 10 feet thick, is a thin layer of clay about 6 inches in thickness, then a second layer of sand rock not as hard as the surface layer. The general construction of the dam is shown by the accompanying diagram showing

a cross-section of the same (page 61). The longitudinal timbers are placed 8 feet from center to center, forming cribs which are filled with stone, as shown by diagram. The bottom timbers of the crib are scribed to fit the ledge (which was first made as level as convenient to receive them), and were bedded in cement mortar and drift bolted to the ledge as indicated on cross-section (page 60), the holes to receive the drift bolts being plugged with wood into which they were driven. The plan of the dam herewith (Plate XIV) shows its location with reference to the crest of the fall. The average depth of water at the site of the dam at the time of beginning construction, was about 2 feet. Work was begun on May 1st, the first coffer-dam having been completed in April. The flow of water was over rapids with a fall of about 2 feet per hundred, and the current due to this slope was the main obstacle to be overcome in the work.

On the north side of the river a simple horse dam was thrown out to sheer the water from that part of the river-bed upon which work was to begin. The horses were first placed in position about 8 feet apart with stringers of 4-inch plank; and beginning at the outer edge of this line of horses, little difficulty was experienced in placing the facing of 2-inch tongued and grooved plank in position, a light footbridge in the rear of the horses being used to carry on the work. A row of bags filled with sand laid on the toe of the sheer dam stopped leakage very effectively, and remained in position in the very swift current. A row of sand-bags laid behind the sheer dam and parallel to it, conducted the leakage away so that the ledge behind was perfectly dry. The sheer dam thus constructed withstood the high water in June, during which month the Upper Missouri reaches its highest stage, the water rising to the top of the planking, and having a depth of about 6 feet on the face of the dam. Work from behind this sheer dam was never delayed, the masonry work proceeded, and a section of the crib work, 150 feet long, was constructed during the period of high water. On the south side of the river a similar course was adopted, to put in the gate chamber masonry, and also a section of the dam 100 feet long. The water on the south side was, however, deeper than on the north, and the grade of the bottom greater, making it necessary to sink small cribs filled with stone to support the wooden horses for the sheer dam. These cribs were triangular in shape, built of 4-inch plank with upright posts, 6

350
340
330
320
310
300
290
MISSOURI
2
10TH
61
47
9TH AVE.
AVE.
26TH ST.
60
48

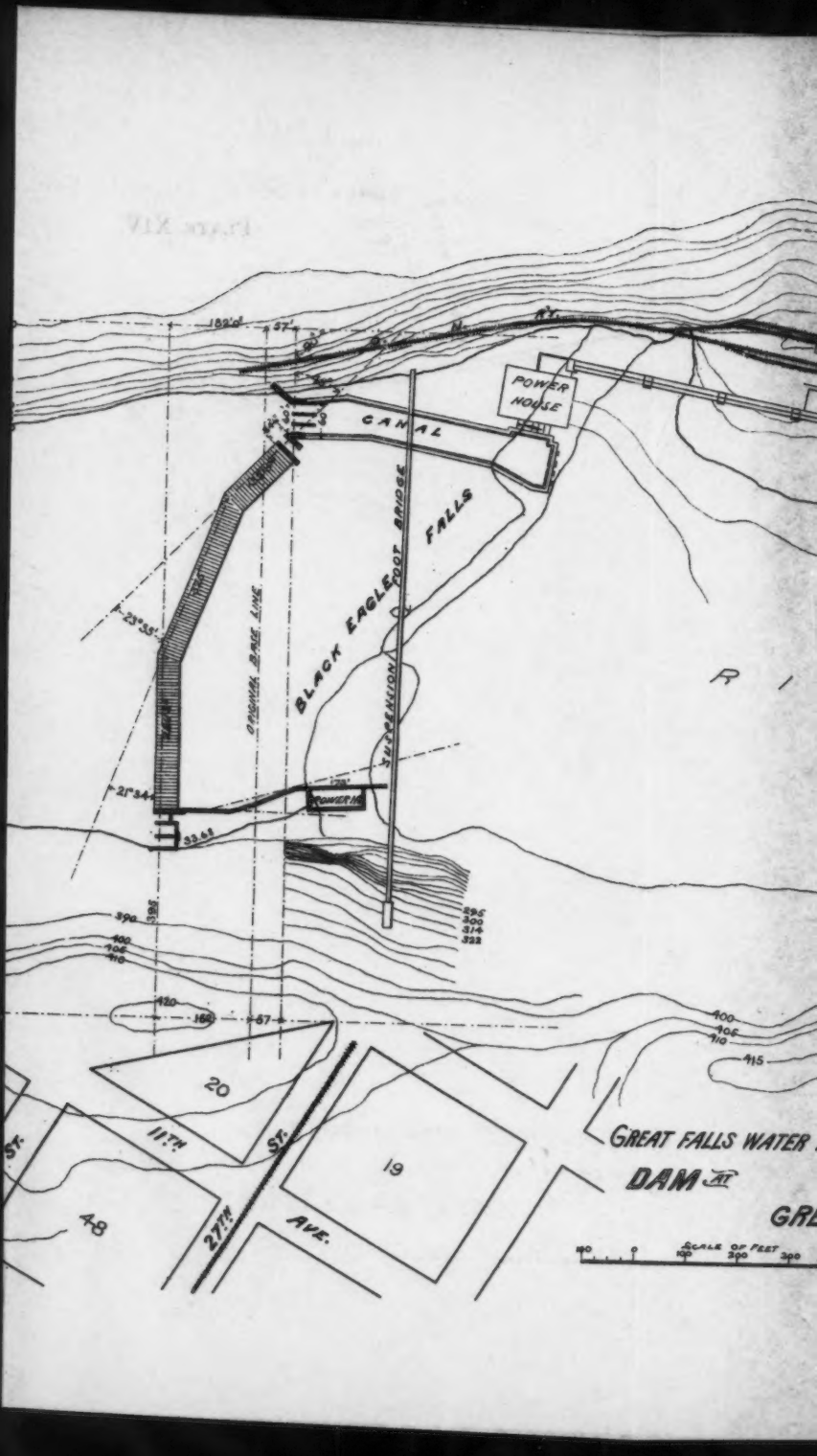
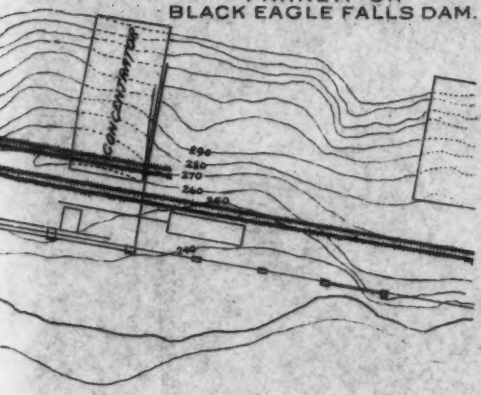
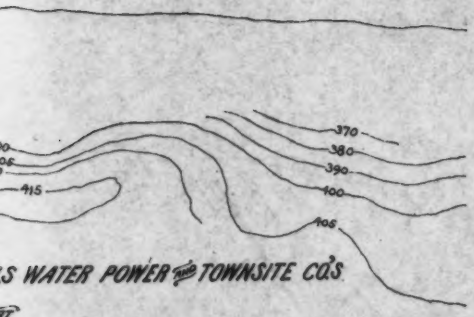

POWER HOUSE
CANAL
BLACK EAGLE DAM
FALLS
PENSTOCK
POWER HS.
23°35'
21°34'
33.63
20
19
48
11TH ST.
27TH AVE.
GREAT FALLS WATER
DAM
SCALE OF FEET

PLATE XIV
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXVII. No. 539.
PARKER ON
BLACK EAGLE FALLS DAM.



I V E R



S WATER POWER & TOWNSITE CO'S

GREAT EAGLE FALLS

SCALE OF FEET 200 300 400 800

J.T. Fanning — CONSLTG. ENGR.
M.S. Parker — ENGINEER.

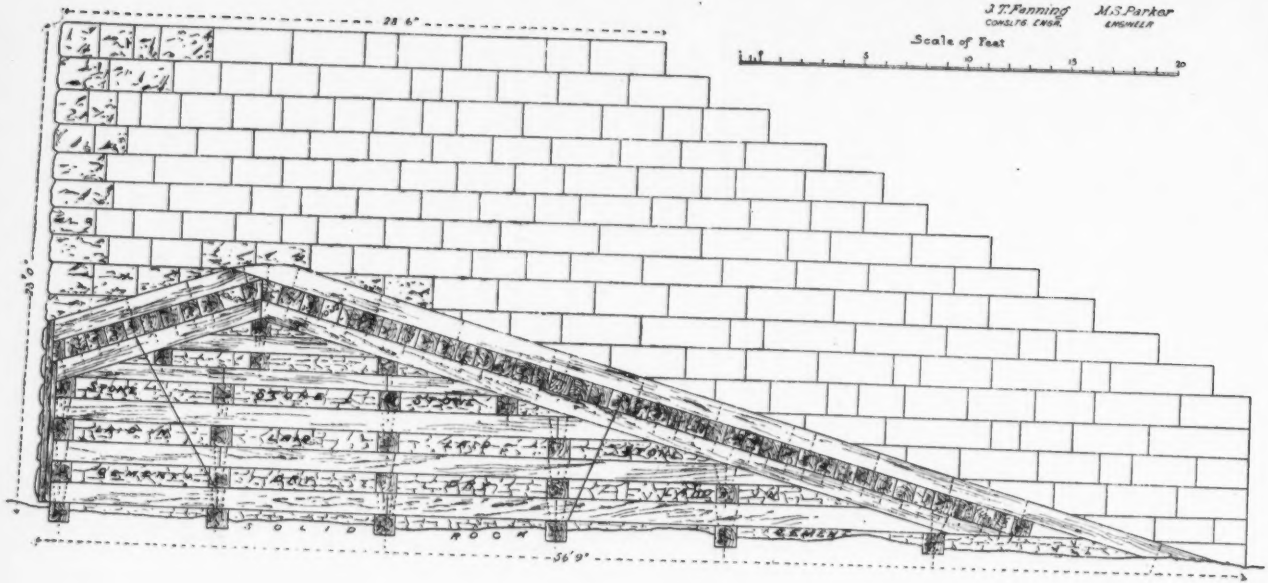
GREAT FALLS WATER POWER AND TOWNSITE CO.

SECTION AT NORTHERLY END OF

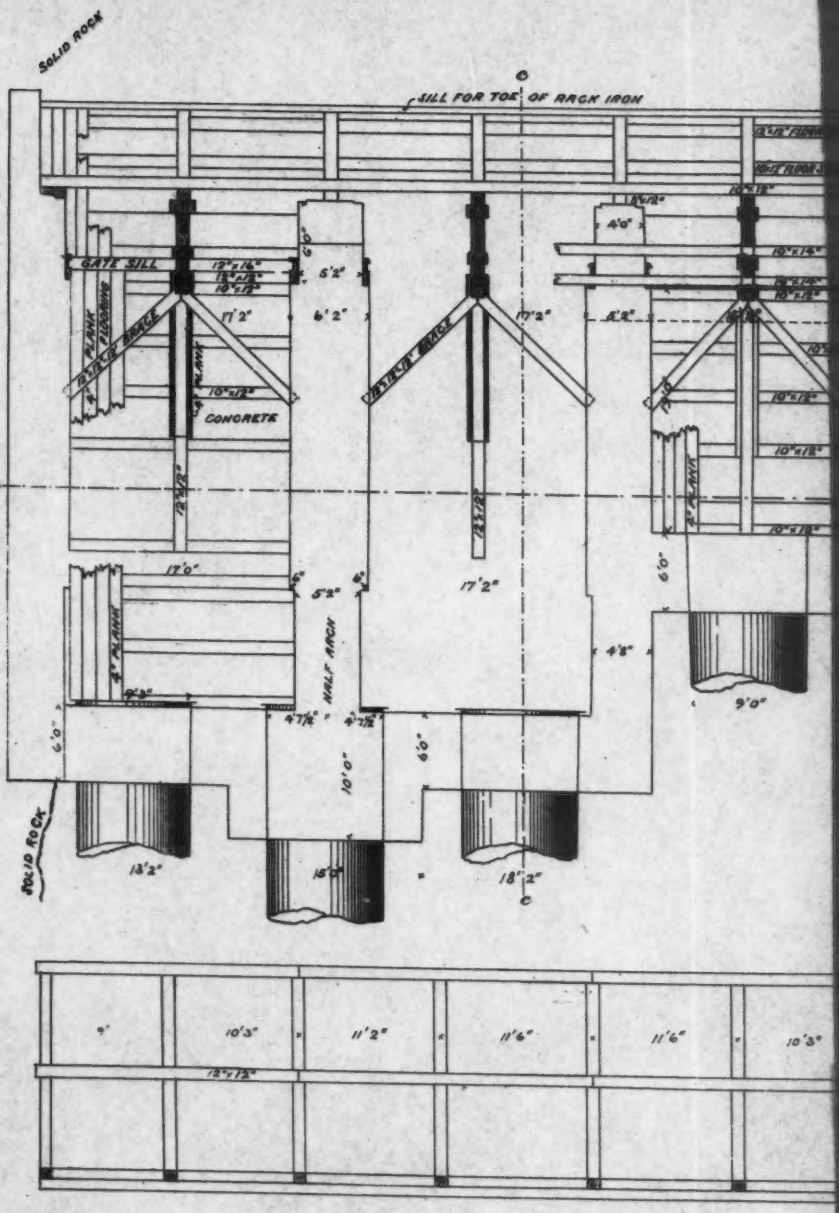
GREAT FALLS DAM

J.T. Fanning, CONSLTG ENGR. M.S. Parker, ENGINEER

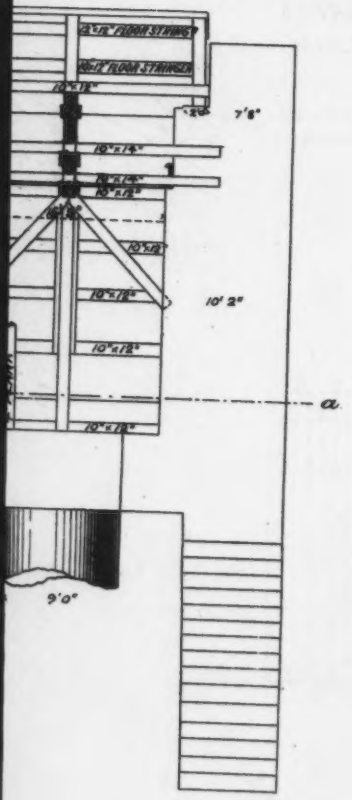
Scale of Feet

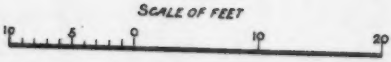


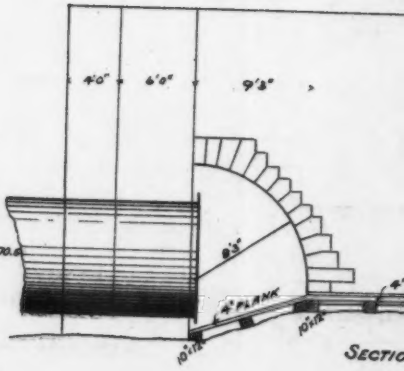
inches square, securely spiked together, were placed 8 feet apart, and worked into position and filled with stone, each crib holding about 1 cubic yard of stone. By the use of a footbridge laid on these cribs, the horses for the sheer dam were carried to position, each crib supporting one horse. No difficulty was experienced in completing this sheer dam, and it was kept in place during the entire time of construction, with the addition of an extra length of matched plank with extra stringers and back supports; it at times carried a depth of 15 feet of water on the face, the leakage being easily controlled with hand pumps (Loud's Non-Chokable Frictionless Pump). Both ends of the dam having been secured, work was begun from the north side only, to close the gap. Between the two ends a footbridge, circular in form, curving up stream, was built. Horse bents were placed 8 feet apart, with a footwalk of 4-inch plank laid on top of them, and upon the caps of these bents, which extended beyond the walk, heavy timber was placed to get the necessary weight to hold them in position. From this bridge wooden horses, similar to those used in the sheer dams, were placed—one in front of each bent of the footbridge—and behind each bent stones were piled, to prevent the structure from sliding. These horses were connected by 4-inch plank stringers, and 4-inch plank was also used for the face of the dam, which was intended only to break the force of the current; this it did admirably. The leakage through this dam was controlled by small sheer dams built of matched plank, and, by the use of bags of sand, the particular section under construction was kept dry. The sections of the crib were 8 feet apart longitudinally; at intervals of about six sections the masonry in front and rear, together with the stone filling in the core, were left out of two sections, leaving an opening 14 feet wide, the entire bottom of which was covered with rubble masonry laid in hydraulic cement up to the top of the bed timbers, the side walls between the timbers being also laid up solid in the same material. The plank for the face of these openings was fitted to the ledge, numbered and piled on top of the dam, to be used when ready to close the openings. As the sections were completed, the cofferdams were removed to allow free passage of water through the openings left; and, before the final closing of the gap in the dam, the gates and chambers at the north end were finished and the eight gates opened, after which all gaps and openings were closed successively. The plank for the face of these openings having been previously fitted,


SOLID ROCK
SILL FOR TOE OF RACK IRON
GATE SILL
PLANK
RACK FLOORING
CONCRETE
HALF ARCH
SOLID ROCK
18'2"
15'0"
18'2"
9'
10'3"
11'2"
11'6"
11'6"
10'3"

GREAT FALLS WATER PO







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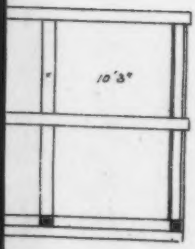
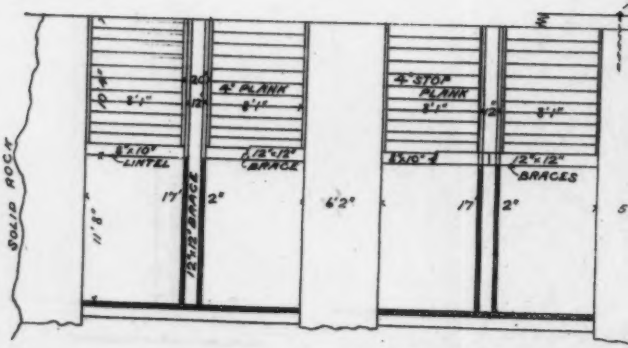




SECTION ON LINE aa

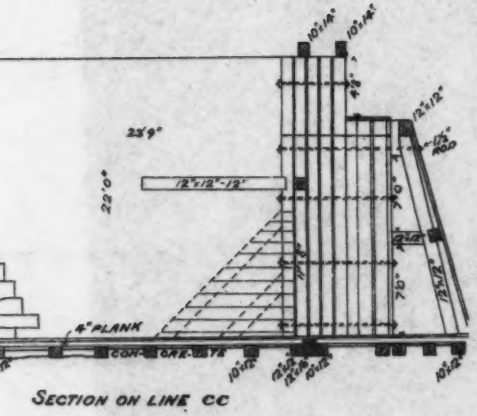
PLATE XV.
TRANS.AM.SOC.CIV.ENGRS.
VOL.XXVII.No 539.
PARKER ON
BLACK EAGLE FALLS DAM.

ER POWER AND TOWNSITE CO.

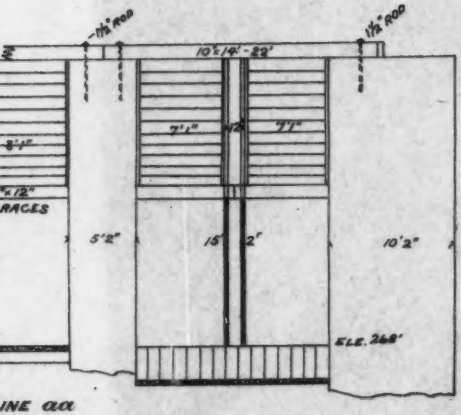
PLAN OF HEAD GATES
SOUTH END OF DAM

J.T. Fanning
CONSLTG. ENGR.

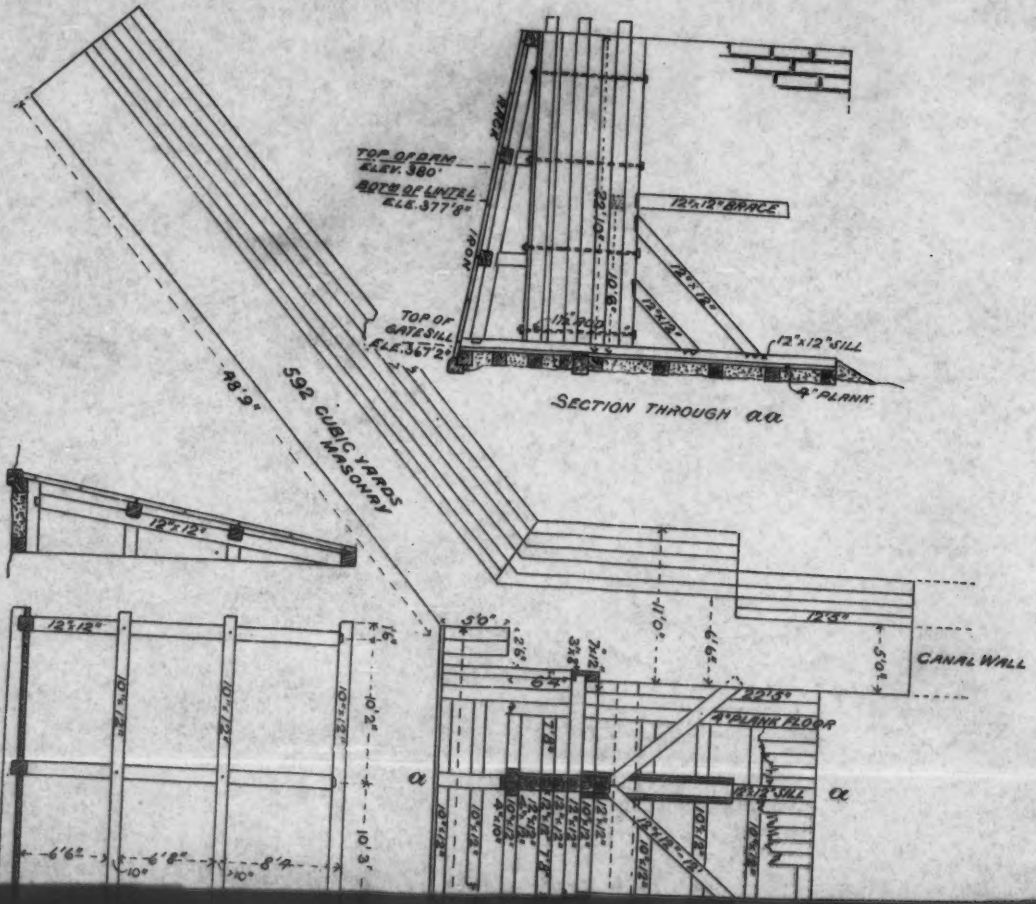
M.S. Parker
ENGINEER



SECTION ON LINE CC



INE aa


TOP OF DAM ELEV. 380'
BOTTOM OF LINTEL ELE. 377'8"
12"x12" BRACE
12"x12"
12"x12"
12"x12" SILL
TOP OF GATE SILL ELE. 367'2"
4" PLANK
SECTION THROUGH aa
48'9"
592 CUBIC YARDS MASONRY
12"x12"
CANAL WALL
4" PLANK FLOOR
12"x12" SILL
a
a
-6'6"-
-6'8"-
-8'4"-
10'3"
10'2"

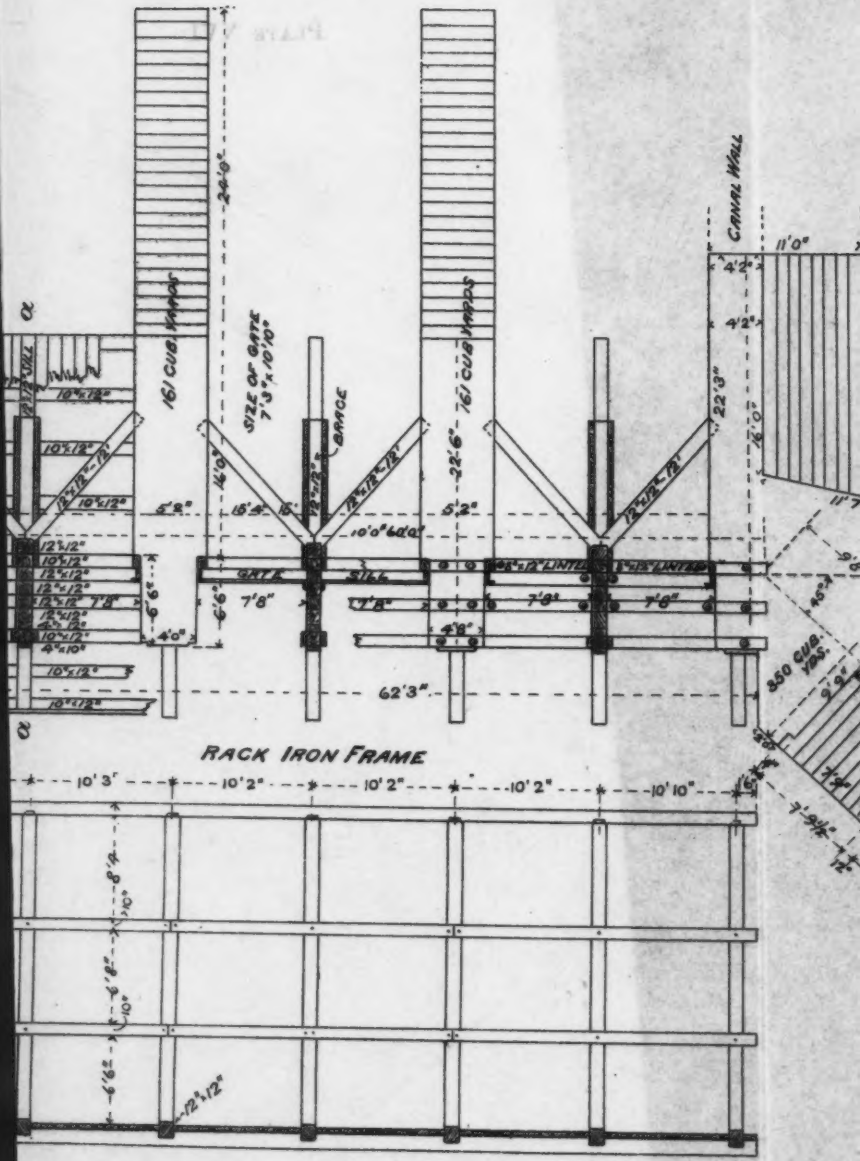

16' CUB. YARDS
SIZE OF GATE 7'3" x 10'10"
BRACE
16' CUB. YARDS
CANAL WALL
11'0"
22'3"
24'0"
14'0"
22'6"
5'2"
15'4"
15'
10'0" 60'0"
62'3"
GATE
SILL
7'8"
4'0"
6'6"
4'2"
350 CUB. YDS.
RACK IRON FRAME
10' 3"
10' 2"
10' 2"
10' 2"
10' 10"
12"x12"

PLATE XVI
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXVII. No. 539.
PARKER ON
BLACK EAGLE FALLS DAM.

GREAT FALLS WATER POWER AND TOWNSITE CO.

PLAN OF WASTE SLUICE AND HEAD GATES, NORTH END OF DAM

SCALE OF FEET
10 5 0 10 20

J. T. Fanning.
CONSLTG. ENGR.

M. S. Parker
ENGINEER.

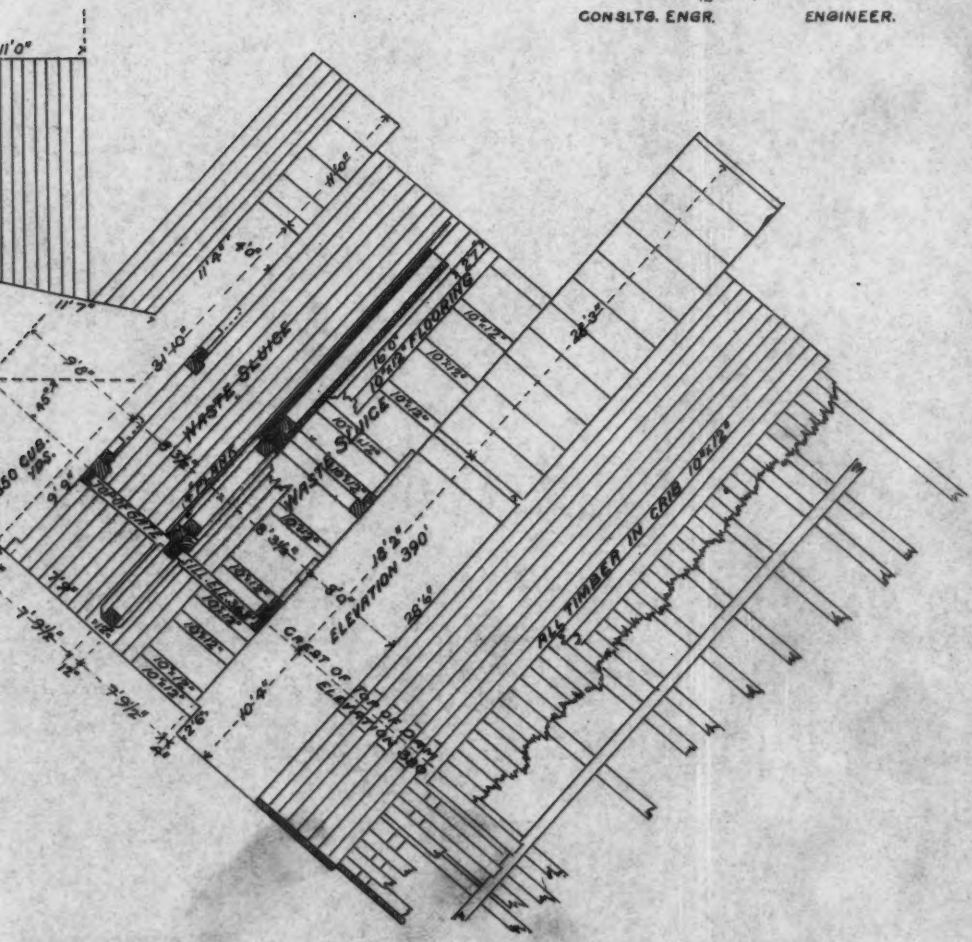


PLATE XVII.
TRANS. AM. SOC. CIV. ENGS.
VOL. XXVII, No. 539.
PARKER ON BLACK EAGLE FALLS DAM.

PLATE XVIII.
TRANS. AM. SOC. CIV. ENGS.
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PARKER ON BLACK EAGLE FALLS DAM.

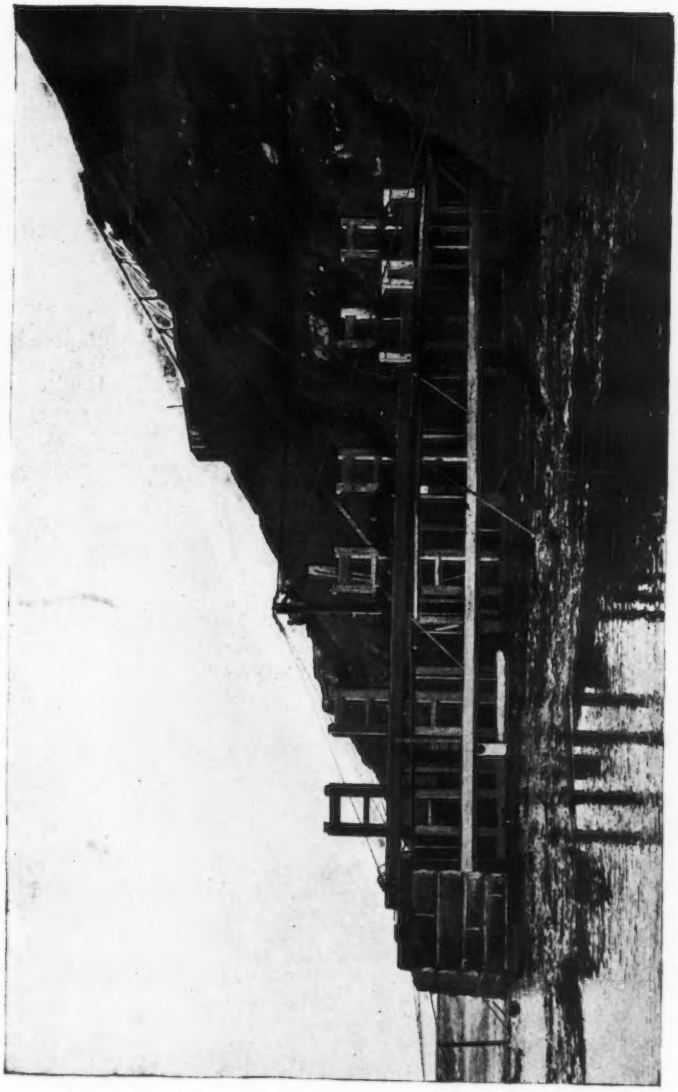

PLATE XIX
TRANS. AM. SOC. CIV. ENGS.
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PARKER ON BLACK EAGLE FALLS DAM.

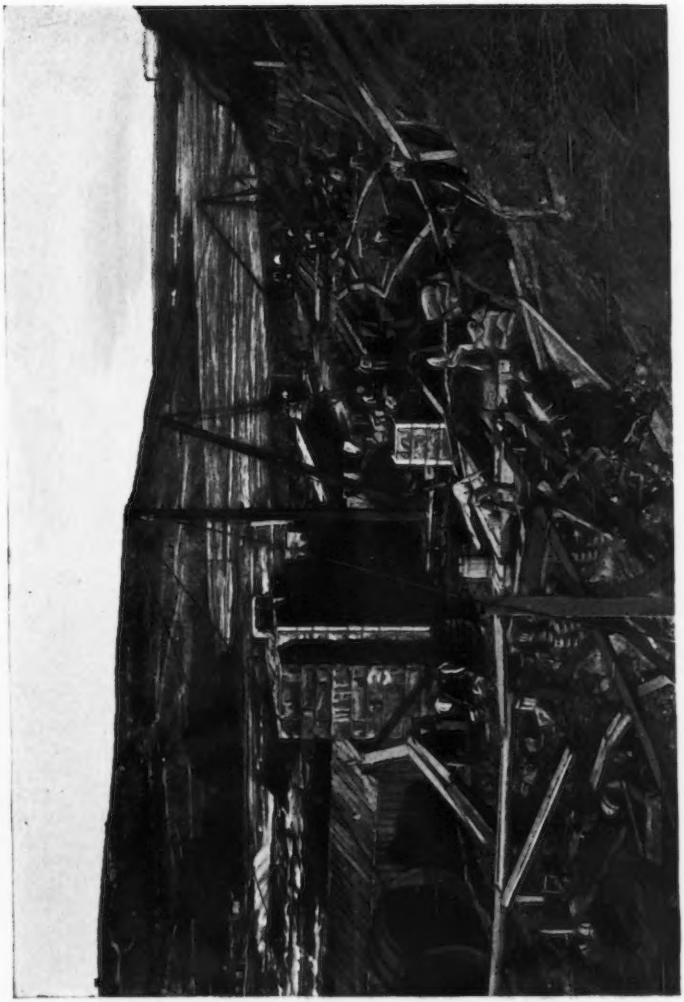


PLATE XX.
TRANS. AM. SOC. CIV. ENGS.
VOL. XXVII, No. 539.
PARKER ON BLACK EAGLE FALLS DAM.

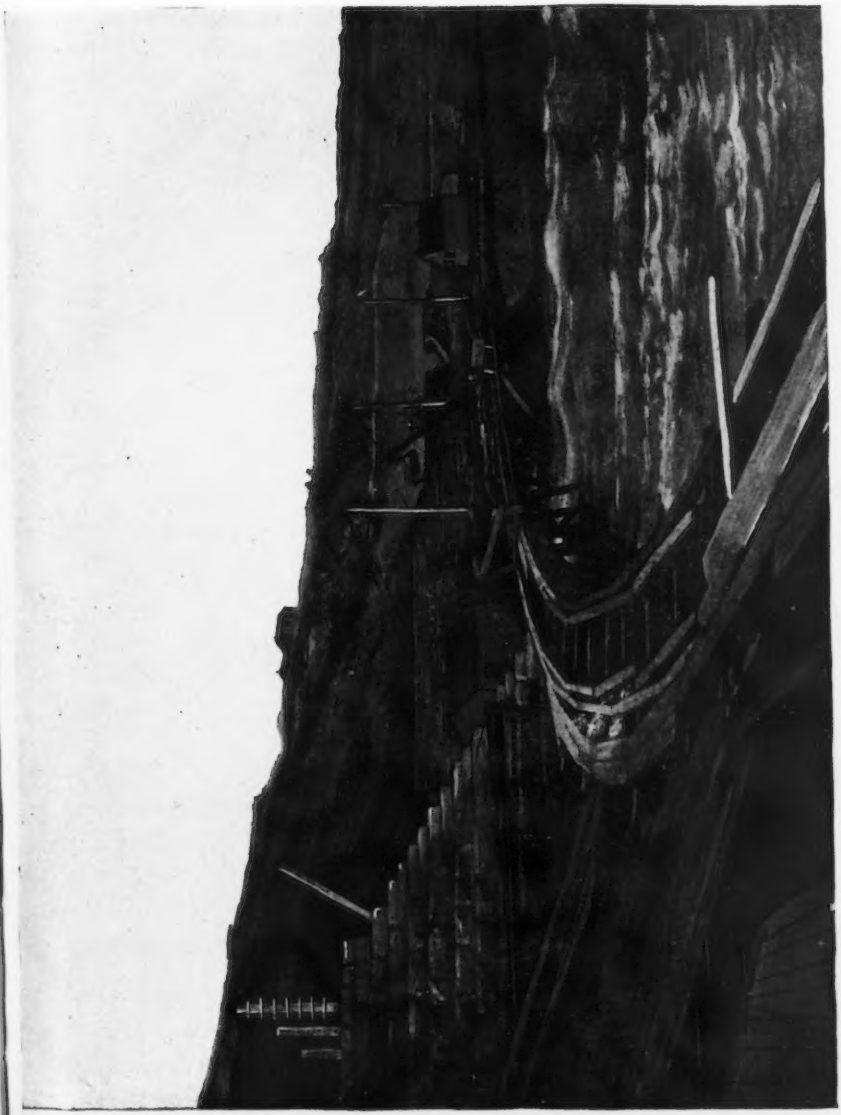
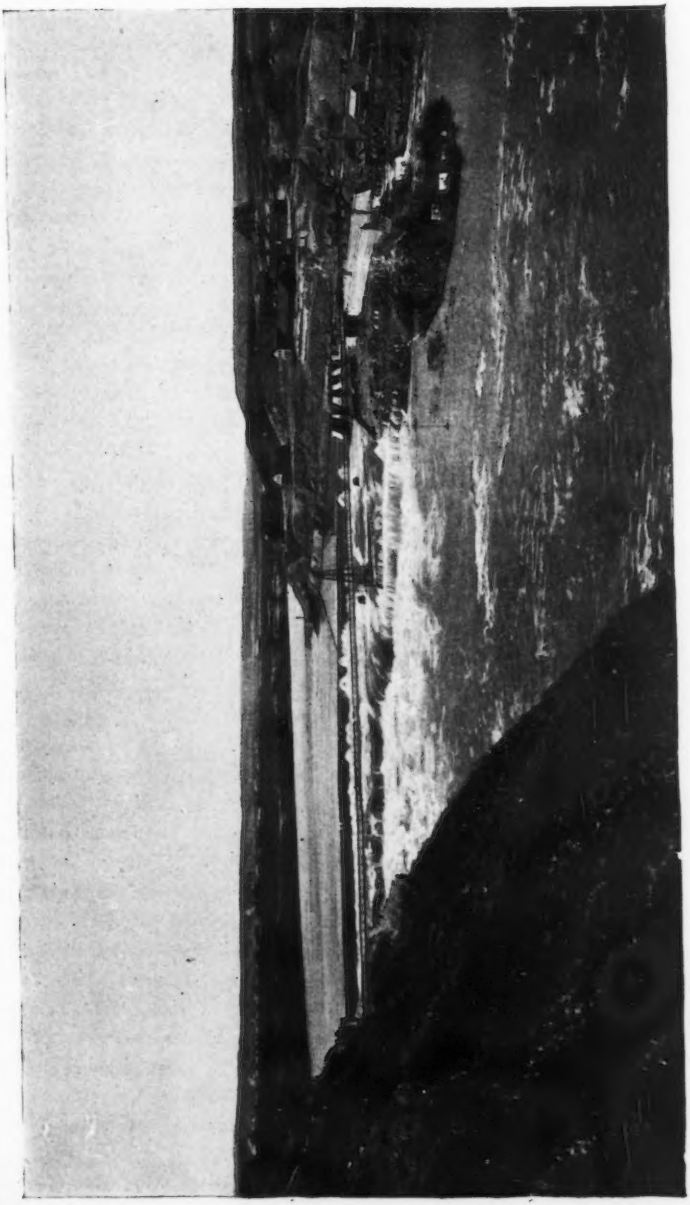


PLATE XXI.
TRANS. AM. SOC. CIV. ENGS.
VOL. XXVII, No. 539
PARKER ON BLACK EAGLE FALLS DAM.



when driven to place, made these compartments free from water, and masonry and stone filling was placed as in the other sections, the entire flow of the river being carried through eight gates, each 7 x 10 feet, and turned out through an opening left in the canal wall for this purpose by a high sheer dam constructed across the canal. This sheer dam was built in the same manner as those before mentioned, excepting that the toes of the horses were bolted to the ledge to prevent sliding.

The accompanying plans and views (Plates XV–XXI) will best illustrate the work as it progressed. The stone in the bed of the river at the dam site is hard but very brittle, and of little use for dimension stone; but, strange to say, there was a body of excellent stone near the middle of the river at the crest of the falls, which proved to be equally as hard as that found a few hundred feet up the river. It could be quarried in blocks of large dimensions, though there was considerable waste in the process. From this point all the stone used in construction was taken. This stone is very heavy, running from 160 to 170 pounds per cubic foot, some specimens running as high as 178 pounds per cubic foot. The average haul of all stone was about 500 feet, push cars being used for transportation. Sand and cement were delivered on both sides of the river at the gate chambers. On the north side a railway track ran directly along the work ; on the south side a spur track delivered the material on the high bluff immediately above the end of the dam, from which it was allowed to slide in shutes to the work below.

The masonry in the dam is laid mostly in American hydraulic cement, Milwaukee and Buffalo cement being used, mixed in the proportion of one part cement and two parts clean sharp sand. Buffalo cement from tests of briquettes from twenty-four hours to three months old, shows an average of about 25 per cent. greater tensile strength than the Milwaukee cement. During the latter part of construction while the weather was cold, English Portland cement was used (Hilton brand), and masonry laid in freezing weather with this cement is now as firm as the rock.

The lumber was all delivered on the north side of the river near the gate chambers. The following table gives the amount of material used and the cost of the labor in construction, and includes the handling of all material after unloading from the cars.

First-class rubble masonry, one-fifth the total cubic yards broken range faced, 4 640 cubic yards, at	$6 56 =	$30 438 40
Cut stone masonry range work, rock faced, close beds and joints, 1 500 cubic yards, at	16 40 =	24 600 00
Stone filling in cribs not laid in cement, 5 000 cubic yards, at	2 10 =	10 500 00
Excavation (no classification), solid rock, loose rock and earth in about equal parts, 10 000 cubic yards, at	1 07 =	10 700 00
Lumber used in dam, 1 200 M. ft. B. M.	10 85 =	13 020 00
Lumber used in gates and chambers, 100 M. ft. B. M.	33 72 =	3 372 00
Engineering expenses, twelve months		5 900 00
Total labor expenses of construction		$98 530 40

Owing to some changes made in the plans of the dam and subsequent location of the power-houses, the original estimate is somewhat at variance with the actual quantities in the work. The work of construction was begun April 15th, 1890, water turned over the crest of dam on January 6th, 1891, and construction completed on March 15th, 1891.

The schedule of wages paid is as follows:

	Per diem.
Common labor	$2 00
Stone masons	4 00
Carpenters	3 50
Quarry men	2 25
Stone cutters	4 50
Quarry foremen	3 50
Masonry "	5 00
Stone cutter foremen	5 00
Carpenter "	4 00

On September 1st the wages of common labor and quarry men advanced twenty-five cents per day.

The price per thousand feet B. M. for gates and gate chambers includes placing of all irons and gearing; the price per thousand for

lumber in the dam includes the placing of iron crest-plate. The expense of false work of all kinds, such as sheer and coffer dams, tramways, bridges, etc., is divided proportionally between the classes of work as given in the table of cost of labor. The tenders called for were for prices on the finished work, and prices given were to cover all false work of whatever description necessary to complete the work.

The total cost of the dam from its inception to November 30th, 1891, including engineer's salary to that date, is $175 000.

The average height of the dam above the ledge is 14 feet, and the crest of the dam is 40 feet above the surface of water below the falls in the tail race.

Assuming the average minimum flow of the river to be 3 360 cubic feet per second (see average flow for October, November and December, 1891), the gross horse-power under 40 feet head is 16 583, and assuming also that 80 per cent. of this power can be utilized on the wheel shafts, we have 12 287 horse-power net, costing $13.19 per horse-power.

The average flow of water over the crest of dam for nine months in the year 1891 by actual gaugings in periods of three months was :

April	..... 12 100	cubic feet per second.
May		
June		
July	..... 7 036	" "
August		
September		
October	..... 3 360	" "
November		
December		

During the first three months of the year the average flow was approximately the same as during the last three months; no gauge, however, was kept during that period. The daily average for the last three months of 1891 would exceed 4 000 cubic feet per second if the days on which anchor ice formed could be eliminated from the record.

The present development of this power consists of two power stations, one of which on the south side of the river is used as an electrical station for power for street railway and electric-light plant. In this station are six Victor turbine wheels developing 1 031 horse-power, four wheels of 20 inches and two wheels of 22½ inches diameter, with space and

connection for two more wheels; wheels running at present in pairs from one penstock 9 feet in diameter; three other penstocks of the same size will be used later for power for other purposes. On the north side of the river a power station developing 2 600 horse-power is located, seven Victor turbine wheels being in use, with two 44-inch, two 22½-inch, two 20-inch and one 40-inch wheel. This plant is for the use of a large smelting works (Boston Montana Consolidated Silver and Copper Smelting Company). In this station there are space and connections for one more penstock, similar to the two now connected. These plants were designed and constructed under the direction of H. A. Herrick, M. Am. Soc. C. E., and are very complete.

The cost of power on a wheel shaft for the first-mentioned power station is $36 per horse-power. This added to the cost of the dam per horse-power, $13.19, makes the total cost $49.19 per horse-power on wheel shaft. Not having a full estimate for the north side station, which is not quite completed, the writer is unable to state the cost, but thinks it will be approximately the same.

Crib dams generally leak badly. Mr. W. W. Follett, Assistant Engineer of the Department of Agriculture, recently read a paper before the Denver Society of Civil Engineers, in which he mentions the Black Eagle Falls Dam and states that it is likely that much of the river's flow is lost by leakage under the structure through crevices in the rock, the larger part of the probable loss being through deep vertical seams in the sandstone of the river bed.

This has not been found to be the case. When the gates were first closed in the few moments that intervened before the water ran over the crest of the dam, the writer observed the foot of the dam closely and found very slight leakage. The river bed below the dam became practically dry, so that people walked about on the ledge without wetting their shoes. There is practically no difference in the river flow over the crest of the dam, and at a point 2 miles above the dam. The theory that the artesian wells of Dakota derive their heads from the upper Missouri River through crevices in the sand rock ledge underlying the river bed, is not borne out by the facts as far as the author was able to observe.

DISCUSSION.

CYRUS C. BABB, Jun. Am. Soc. C. E.—Records of the daily gauge heights of the Missouri River at Craig, about 50 miles above Great Falls, Mont., have been maintained by the United States Geological Survey for a period of two years. These, combined with the measurements of the discharge of the river at the same place, give the daily discharge for this period. The drainage area of the river at the gauging station is 17 615 square miles, while at Great Falls, below the mouth of the Sun River, it is 23 540 square miles.

MISSOURI RIVER, CRAIG, MONT.

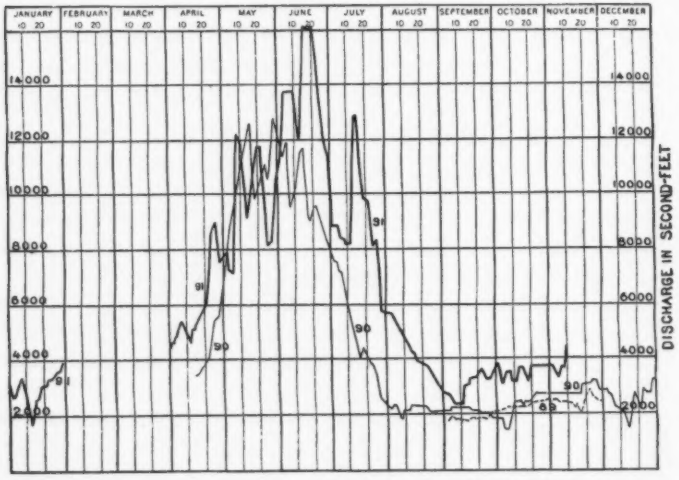
Drainage Area, 17 615 *Square Miles.*

MONTH.	DISCHARGE IN SECOND-FEET.			Total for Month in Acre-Feet.	RUN-OFF.	
	Maximum.	Minimum.	Mean.		Depth in Inches.	Sec.-ft. per sq. mile.
January, 1890	.....	.....	*3 000	184 500	0.20	0.17
February, "	.....	.....	*3 000	166 500	0.18	0.17
March, "	.....	.....	*3 000	184 500	0.20	0.17
April, "	6 100	3 595	4 662	277 389	0.30	0.26
May, "	12 500	6 900	10 472	644 030	0.68	0.59
June, "	11 900	8 100	10 074	599 401	0.64	0.57
July, "	7 800	2 614	5 020	308 730	0.33	0.28
August, "	2 505	1 960	2 216	136 284	0.14	0 13
September, "	2 396	1 960	2 232	132 804	0.14	0.13
October, "	2 723	1 742	2 379	146 308	0.16	0.13
November, "	3 159	2 723	2 868	170 646	0.18	0.16
December, "	3 159	1 742	2 763	169 924	0.18	0.16
January, 1891	3 823	1 742	2 967	182 470	0.19	0.17
February, "	.....	.....	*3 500	215 250	0.24	0.20
March, "	.....	.....	*4 000	246 000	0.26	0.23
April, "	9 130	4 576	5 794	344 743	0.37	0.32
May, "	12 050	7 150	9 015	554 422	0.59	0.51
June, "	16 355	11 000	13 645	811 877	0.85	0.77
July, "	12 960	5 890	9 115	560 572	0.60	0.52
August, "	5 890	2 832	4 415	271 522	0.29	0.25
September, "	3 704	2 614	3 078	183 141	0.19	0.17
October, "	3 704	3 159	3 511	215 926	0.23	0.20
November, "	4 358	3 486	3 802	226 219	0.25	0.22
December, "	.....	.....	*3 200	196 800	0.21	0.18

* Estimate.

The only tributaries of importance entering between these two places are the Dearborn, Smith and Sun Rivers. The average summer flow of the Dearborn is 50 second-feet. Very little water finds its way into the Missouri from Smith River except in times of flood. The discharge of the Sun River above Augusta for 1890 was 715 second-feet, with an average summer flow of 200 second-feet. Most of the water of this river is, however, appropriated before it reaches the Missouri.

The following table gives the monthly flows of the Missouri River at Craig, Mont., from April 17th, 1890, to December 31st, 1891. The first column gives the month; the second, third and fourth give respectively the maximum, minimum and mean discharge in cubic feet per second, or second-feet as it is commonly called; the fifth, the total flow of the month in acre-feet, an acre-foot being the amount of water covering one acre one foot in depth, or 43 560 cubic feet. The last two columns express the run-off in two ways: the sixth gives the depth that a plain of the size of the drainage basin at this place would be covered if the total flow for the month were spread over it; the seventh and last column gives the second-feet per square mile of area drained.



MISSOURI RIVER AT CRAIG, MONTANA.

The mean annual discharge for 1890 was 4 307 second-feet, giving a depth of annual run-off of 3.3 inches; for 1891 the corresponding figures are 5 503 second-feet and 4.4 inches. The average flows for the last five months of each year, or practically the low-water discharges, are 2 500 and 3 600 second-feet respectively.

In the appended graphical representation of the daily discharge, the characteristics of this river are shown, with its low-water discharge for the first three months of the year, its sudden rise in the latter part of April, with a continuance of the higher level for about a month, and then a gradual decline through the months of July and August, with a low-water discharge for the rest of the year. The line for 1889 in the diagram is the discharge of the river for that year at Canyon Ferry.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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540.

(Vol. XXVII.—July, 1892.)

A NEW FORMULA FOR THE STRENGTH OF COLUMNS.

By A. J. DUBOIS, Jun. Am. Soc. C. E.

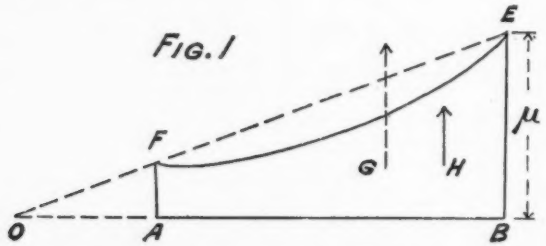
WITH DISCUSSION.

The writer wishes to present a new formula for the strength of columns, which, in so far as he has been able to test it, seems to be more accurate and to possess wider scope than any of those in common use. The derivation of the formula will be given briefly and then the applications of it which lead him to think it valuable. The object is, of course, to have it further tested by experimental results. It can, at least, be shown in this paper that it is worth while to thus test it thoroughly. It may be said, in advance, that the formula contains no empirical constants to fit it to any set of experiments, but depends only upon the length, the cross-section, the physical qualities of the

material and the end conditions. It would, therefore, seem that if it gives good results in a few widely separated cases, it ought to give equally good results in others also. Actual comparison of its results with the results of experiment must, however, settle this question. Such comparisons are, therefore, given as the writer has been able to make and additional ones are invited.

Derivation.—Let μ be the breaking compressive unit stress for the material, as determined by experiment upon very short specimens.

Let AB, Fig. 1, be the cross-section of a column at the point where the compressive unit stress in the outer fiber is greatest, and suppose the column to be at the point of rupture, so that this compressive unit stress in the outer fiber is the breaking unit stress μ. Let $BE = \mu$ represent this stress.



At the point of rupture the limit of elasticity is greatly exceeded, and, as we know, the fiber forces cannot be represented by the ordinates to a straight line, as is assumed in the common theory of flexure. Let, therefore, the ordinates to the curve FE represent the fiber forces.

Now, we know that for very long columns with round ends, the breaking load as given by experiment, is very accurately given by "Euler's formula," viz.:

$$\frac{P}{A} = nE\frac{r^2}{l^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where A is the area of cross-section in square inches; P the crippling load in pounds; l the length of column and r the least radius of gyration of the cross-section, both in inches; E the coefficient of elasticity in pounds per square inch; and $n = \pi^2$ for round ends.

But this equation (1) is deduced by the common theory of flexure, upon the explicit assumption that the limit of elasticity is not exceeded.

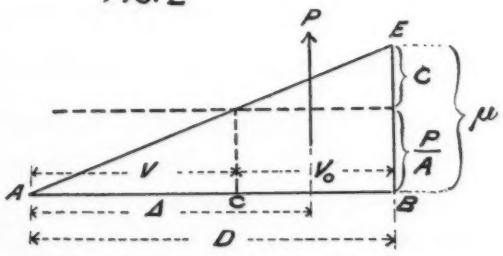
That is, the fiber forces are assumed to be represented by ordinates to the straight line FE (Fig. 1).

How then can it happen that the result of this erroneous assumption is found to be practically correct for experiments made at the breaking point? One reason why this is so can be seen by an examination of Fig. 1. The curve FE represents the actual fiber forces, the center of action of their resultant being represented by the arrow at H. The straight line FE represents the fiber forces as assumed by the theory of flexure. We see that the resultant of these is greater but acts at a point G nearer the neutral axis O. The moment, therefore, with respect to O, may be the same for both resultants, and actual experiment seems to indicate that the compensation is very close, and, hence, that so far as this is concerned, the common theory of flexure leads to results which hold good at the breaking point, and we can replace the curve FE by the straight line FE.

As the length of the column increases, the compressive fiber stress, other things being unchanged, will increase on the concave side and decrease on the convex side, and the neutral axis O approaches A in Fig. 1.

Evidently we may have such a ratio $\frac{l}{r}$, that when BE becomes μ, AF shall be zero, and the neutral axis coincide with A. We shall then have Fig. 2.

FIG. 2

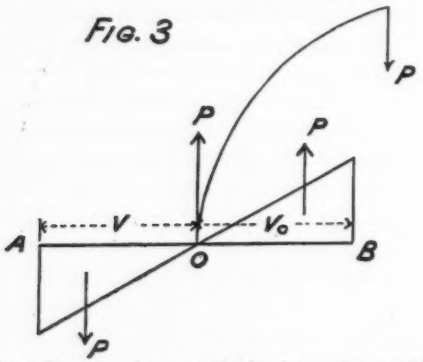


If l now increases, we shall have compression on one side and tension on the other, since the neutral axis will be on the right of A. In other words, pure flexure occurs.

Moreover, since a couple can only be held in equilibrium by another couple, the moment the neutral axis passes to the right of A in Fig. 2,

it will not stop until the condition of Fig. 3 is reached, and the neutral axis is at the center of gravity of the cross-section, AB.

FIG. 3



That is, the column springs suddenly from the condition of Fig. 2 to that of Fig. 3. This is a phenomenon often observed in experimental tests on long columns. We have in Fig. 2, then, the condition of things at the limit when pure flexure is about to begin.

Values of n.—Now, if we suppose the column to have flat ends instead of round, we can find the value of n in equation (1), by the proportion $n : \pi^2 :: v + \frac{2}{3} v_o : \frac{2}{3} v_o$, hence for flat ends generally,

$$n = \left(\frac{3\,v}{2\,v_o} + 1\right) \pi^2 \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where v and v_o are the distances from the center of gravity of the cross-section AB to the outer fiber on the convex and concave sides respectively.

For symmetrical cross-sections, $v = v_o$, and for flat ends, symmetrical cross-section,

$$n = \tfrac{5}{2}\,\pi^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

These values of n substituted in equation (1), ought to give $\frac{P}{A}$ for flat ends, after pure flexure has taken place.

For pin ends, we know by experiment that $\frac{P}{A}$ increases with the diameter of the pin. If we assume that when the diameter of the pin d is equal to $AB = D$, the breaking strength is equal to that for flat ends, we can write for pin ends, generally,

$$n = \tfrac{2}{3}\,\frac{\left(D + \frac{d}{2}\right)}{D}\left(\frac{3\,v}{2\,v_o} + 1\right)\pi^2 \ldots\ldots\ldots\ldots (4)$$

For symmetrical cross-sections, $v = v_o$, and this becomes for pin ends, symmetrical cross-section,

$$n = \tfrac{5}{3} \frac{\left(D + \frac{d}{2}\right)}{D} \pi^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

This approaches the limit $\frac{5}{3} \pi^2$ as the pins diminish.

For one pin and one flat end, we may take the mean of (2) and (4), and have for one pin and one flat end, generally,

$$n = \frac{\left(5\,D + \frac{d}{2}\right)}{6\,D} \left(\frac{3\,v}{2\,v_o} + 1\right) \pi^2 \ldots\ldots\ldots\ldots\ldots (6)$$

For symmetrical sections this becomes

$$n = \tfrac{5}{2} \frac{\left(5\,D + \frac{d}{2}\right)}{6\,D} \pi^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

This approaches the limit $\frac{25}{12} \pi^2$ as the pin diminishes.

Value of $\frac{P}{A}$ *for Pure Flexure.*—We have also from Fig. 2,

$$\mu = C + \frac{P}{A}$$

where C is the unit compression due to flexure alone, and $\frac{P}{A}$ is the direct compression due to the load P.

From the theory of flexure we have

$$P\,\Delta = \frac{CA\,(r^2 + v^2)}{v} \text{ or } C = \frac{P\Delta\,v}{A\,(r^2 + v^2)}$$

where r is the least radius of gyration of the cross-section and Δ the deflection.

Substituting this value of C, we have

$$\frac{P}{A} = \frac{\mu}{1 + \frac{\Delta v}{r^2 + v^2}}$$

But we see from Fig. 2, that $\Delta = \frac{2}{3} D$. We have then for the point where pure flexure begins,

$$\frac{P}{A} = \frac{\mu}{1 + \frac{2\,Dv}{3\,(r^2 + v^2)}} = \frac{\mu}{1 + e} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

where $e = \frac{2\,Dv}{3\,(r^2 + v^2)}$.

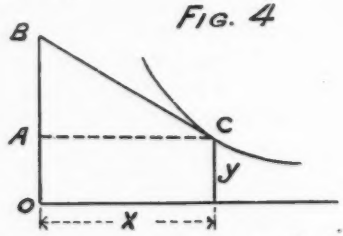
For rectangular cross-sections .. $e = 1$ and $1 + e = 2$
" solid round " " .. $e = 1\frac{1}{15}$ " $1 + e = 2\frac{1}{15}$
" hollow round " " .. $e = \frac{8}{9}$ " $1 + e = 1\frac{8}{9}$

For other cross-sections, e can be easily computed in any given case.

We see, however, that it does not differ much from 1, and hence, in general, $\frac{P}{A}$ at the point where pure flexure commences is not far from $\frac{\mu}{2}$.

Johnson's Straight Line Formula.—In the *Transactions* Am. Soc. C. E. for July, 1886, Mr. Thomas H. Johnson, M. Am. Soc. C. E., presented his now well-known "straight line formula." He pointed out the property of Euler's curve, viz., that a tangent to this curve at any point cuts off on the y axis a distance equal to three times the ordinate at the point of tangency. Thus in Fig. 4, BC is a tangent to the curve at C, $OB = 3y$.

FIG. 4



This is easily demonstrated as follows: Let $\frac{P}{A} = y$ and $\frac{l}{r} = x$, Fig. 4. Then equation (1) becomes

$$y = \frac{nE}{x^2}.$$

Differentiating, we have for the tangent of the angle which BC makes with the horizontal

$$\frac{dy}{dx} = -\frac{2nE}{x^3}.$$

Hence, the distance AB, Fig. 4, is $-x\frac{dy}{dx} = \frac{2nE}{x^2} = 2y$. The distance OB is therefore $3y$, and the abscissa OE to the point of tangency is $OE = x = \sqrt{\frac{3nE}{OB}}$

Now the portion BC, Fig. 4, is Mr. Johnson's straight line. By plot-

ting all the experiments available, he determined OB for wrought iron at 42 000 pounds per square inch, and found for n, for flat ends, $n = \frac{5}{2}\pi^2$, and for pin ends $n = \frac{5}{3}\pi^2$.

These latter values agree with those we have already deduced in (3) and (5), for symmetrical sections and small pins. It is believed, as the result of these investigations, that the values given by (2) and (4) are more general.

Mr. Johnson's formula is then—

$$\text{for } \frac{l}{r} < \sqrt{\frac{3\,nE}{OB}}$$

$$\frac{P}{A} = \overline{OB} - \frac{2}{3}\overline{OB}\sqrt{\frac{\overline{OB}}{3\,nE}}\,\frac{l}{r}$$

$$\text{for } \frac{l}{r} \geq \sqrt{\frac{3\,nE}{OB}}$$

$$\frac{P}{A} = \frac{nEr^2}{l^2}$$

where n depends on the end conditions as already stated. These formulas have been widely adopted, and in the paper of Mr. Johnson referred to, he showed that they gave good average results, taking all the experiments together.

Mr. Johnson's value of $\overline{OB}$ was, however, about $\frac{2}{3}\,\mu$. This fact was commented upon by Mr. James Christie, M. Am. Soc. C. E.

The Proposed Formula.—Like Mr. Johnson, we propose—

$$\text{for } \frac{l}{r} \geq \sqrt{\frac{3\,nE}{\mu}}$$

$$\frac{P}{A} = \frac{nEr^2}{l^2} \qquad \text{(9)}$$

It will be observed that OB, Fig. 5, is taken equal to the full value of μ in any case, not to the average for a number of cases, nor to any fraction of the average.

As these values for n agree practically with Mr. Johnson's, both give the same formula (9) for large values of $\frac{l}{r}$, except that the limit here given is a little different. Thus, for wrought iron he would take $\mu = 42\,000 = oB^1$, Fig. 5, while the writer would take μ, whatever its full value for the material may be, which for wrought iron would in general be nearly 60 000. This gives a different location for the point E, Fig. 5.

A straight line from C to D, Fig. 5, is then drawn, the ordinate at D

being $\frac{\mu}{1+e}$, where e has been already found from (8), and shown in most cases to be not far from $e = 1$.

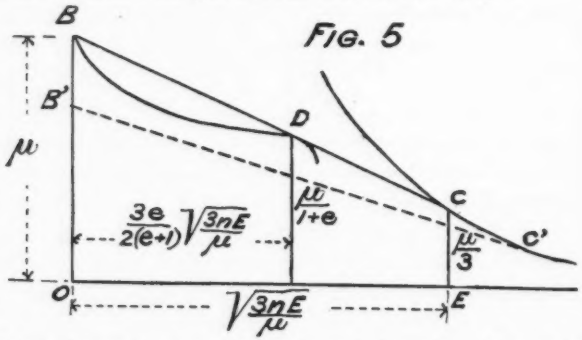
We therefore have for medium values of $\frac{r}{l}$,

$$\text{for } \frac{l}{r} > \frac{3\,e}{2\,(e+1)} \sqrt{\frac{3\,nE}{\mu}} \text{ and } < \sqrt{\frac{3\,nE}{\mu}}$$

$$\frac{P}{A} = \mu \left[1 - \frac{2\,l}{3\,r} \sqrt{\frac{\mu}{3\,nE}} \right] \quad \ldots\ldots\ldots\ldots\ldots \quad (10)$$

This differs from Mr. Johnson's straight line in two important respects. First, its point of tangency C, Fig. 5, and its inclination depend upon the actual value of μ in any case. Second, it stops at D, while Mr. Johnson's straight line runs from C^1 to B^1, Fig. 5.

FIG. 5



This is the weak point of Mr. Johnson's formula. All experiments the writer has thus far been able to plot, show from D to B a curve similar to that shown in Fig. 5. This curve always runs under the straight line DB, and it includes many of the most commonly occurring ratios of $\frac{l}{r}$.

It is for this reason that Mr. Johnson was obliged to run his straight line lower down, making OB^1 less than μ. This strikes an average, but follows no given set of experiments.

For the curve from D to B, Fig. 5, the writer has, after much labor and testing, assumed the form $y = \mu\,[1 - ax - bx\,(1 - bx)^2]$

Determining the constants a and b by the conditions that the curve shall be tangent to the line DC at D, we easily find—

$$a = \frac{2}{3} \sqrt{\frac{\mu}{3\,nE}}, \quad b = \frac{2\,(e+1)}{3\,e} \sqrt{\frac{\mu}{3\,nE}}$$

We have then—

$$\text{for } \frac{l}{r} = 0 \text{ up to } \frac{l}{r} = \frac{3\,e}{2\,(e+1)}$$

$$\frac{P}{A} = \mu\left[1 - \frac{2\,l}{3\,r}\sqrt{\frac{\mu}{3\,nE}} - \frac{2\,(e+1)\,l}{3\,er}\sqrt{\frac{\mu}{3\,nE}}\right.$$

$$\left.\left(1 - \frac{2\,(e+1)\,l}{3\,er}\sqrt{\frac{\mu}{3\,nE}}\right)^2\right] \ldots\ldots\ldots\ldots\ldots (11)$$

It will be seen then, that our proposed formula comprises Euler's curve (9) for large values of $\frac{l}{r}$, with the values n given by (2), (4) and (6); a straight line given by (10) for intermediate values of $\frac{l}{r}$; and a curve given by (11) for medium and small values of $\frac{l}{r}$. The limits are determined by the ordinates $\frac{\mu}{e+1}$ and $\frac{\mu}{3}$. The value of e is given by

$$e = \frac{2\,Dv}{3\,(r^2+v^2)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

where D is the breadth of cross-section in the direction of bending, v is the distance from center of gravity of the cross-section to the outer fiber on the tension side, r is the radius of gyration of the cross-section —all in inches.

Nearly all practical cases lie within the limits of (11), and within these limits it seems certain from all the experiments the writer has been able to obtain, the law is not that of a straight line.

Equations (9), (10) and (11), it will be observed, contain no empirical constants, to fit them to any special set of experiments. The constants μ and E depend upon the material and can be experimentally determined on short specimens. The constant n depends only upon the end conditions as given by (2), (4) and (6), and the constant e depends upon the shape of cross-section only as given by (12). The formulas depend therefore only upon the length, the cross-section, the physical qualities of the material and the end conditions, in any given case.

Application of the Formulas.—In the *Transactions* Am. Soc. C. E. for January, 1882, will be found a series of experiments made upon full-sized wrought-iron Phœnix columns, with the United States Government testing machine at Watertown (Mass.) Arsenal, presented by T. C. Clark, A. Bonzano, John Griffen and David Reeves, Members Am.

Soc. C. E. In Table No. 1 we give the results of these experiments. The columns were all flat ends. According to the data given, we have $D = 8$ inches, $v = 4$ inches, $r = 3$ inches, and hence from (12) $e = 0.853$. We see also from the experiments that μ is not far from 57 000 pounds per square inch.

We are not told the value of E. We assume it at 33 960 000 pounds per square inch. A considerable change in its value between the limits of 38 000 000 and 16 000 000 will cause but little variation in the results of our formula (11) for the range of experiments given. We have chosen the value of E which is indicated by our formula.

We have then $n = \frac{5}{2}\pi^2$ from (3) and $\sqrt{\frac{3nE}{\mu}} = 210$.

Hence $\frac{2}{3}\sqrt{\frac{\mu}{3nE}} = \frac{1}{315}$ and $\frac{2(e+1)}{3e}\sqrt{\frac{\mu}{3nE}} = \frac{1}{145}$.

Between the limits of $\frac{l}{r} = 0$ and $\frac{l}{r} = 145$, therefore, we have from equation (11),

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{315\,r} - \frac{l}{145\,r}\left(1 - \frac{l}{145\,r}\right)^2\right]$$

and this equation includes the entire range of the experiments.

In Table No. 1 are given the results of this formula in the last column, and of the experiments in the next to the last column.

These experiments have been plotted in Plate XXII, Fig. 1, the formula curve is drawn in a full line, and the "straight line formula" of Mr. Johnson, and the Rankine-Gordon formula in broken lines. The Rankine-Gordon formula adopted is

$$\frac{P}{A} = \frac{40\,000}{1 + \frac{l^2}{36\,000\,r^2}}$$

A glance at the plate shows that the formula proposed follows the law of the experiments much more closely than either of the other formulas. It is also evident that the last two do not express this law at all. All that can be said of them is that they are certainly safe.

By reference to Table No. 1, it will be seen that the experimental values themselves vary considerably in some cases, even for the same area, ratio $\frac{l}{r}$, length and r. Such deviations may be due to lack of exact centering, lack of perfect straightness, lack of uniformity of E for compression, etc.

FIG. 1.

Watertown Experiments. Clark, Reeves & Co.

Phoenix Columns, Flat Ends, Wrt. Iron.

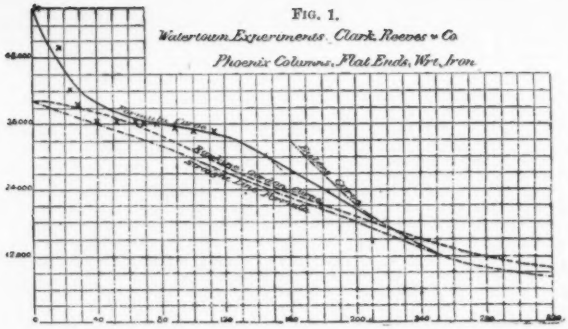


FIG. 3.

Lindley's Experiments

Pin Ends - Channels - Wrt. Iron.

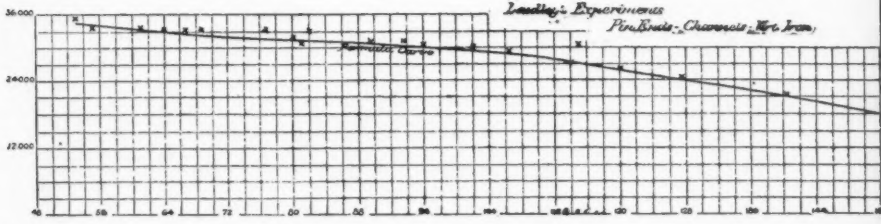


FIG. 4.

Tubular and Solid Rectangular

Flat Ends - Wrt. Iron.

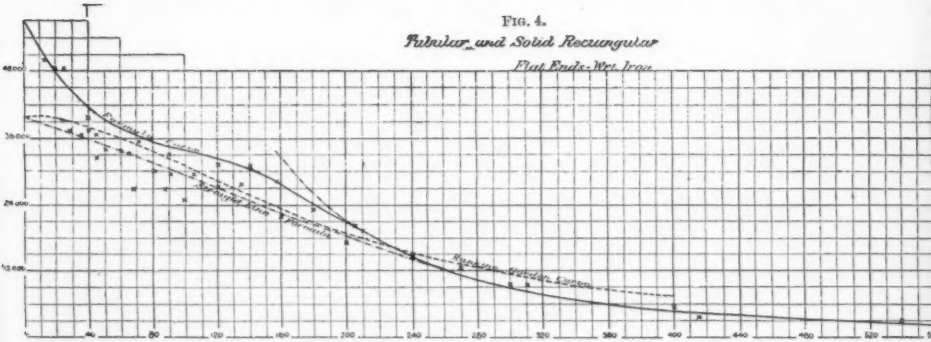


FIG. 6.

Christie's Experiments. Flat Ends - Wrt. Iron.

Angles (r), Tees (•), Channels (x) and Tubes (o)

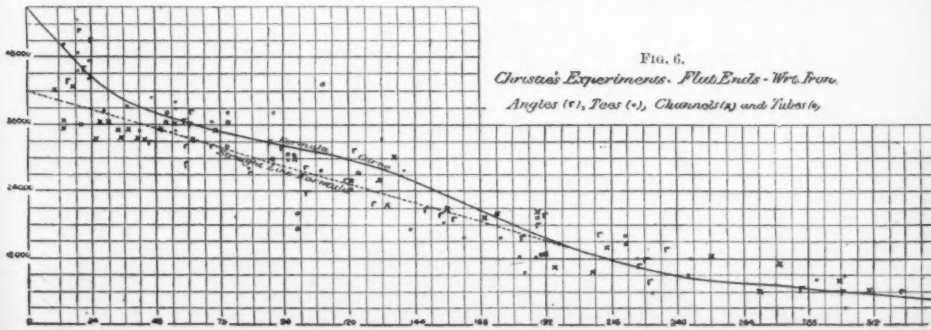
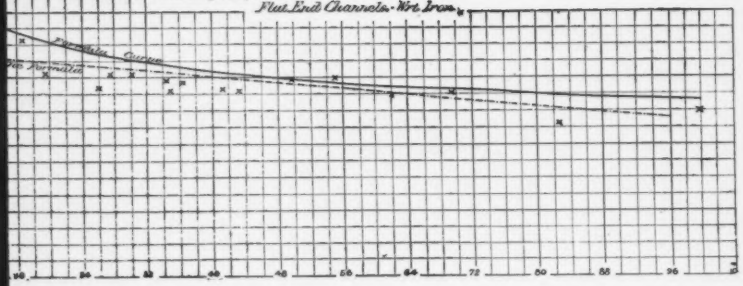
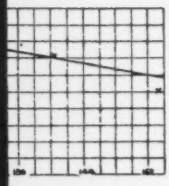


FIG. 2.
Laidley's Experiments.
Flat End Channels.-Wt. Iron.





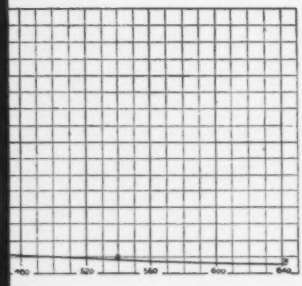
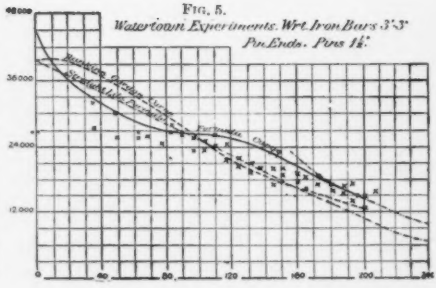
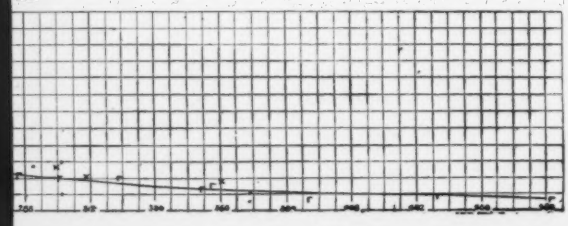


FIG. 5.
Watertown Experiments. Wt. Iron Bars 3"×3"
Pin Ends. Pins 1¼".





A formula, therefore, which accurately expresses the strength of a column in terms of the known physical properties only, should be expected to show variations from the results of experiment. The character of these variations, however, ought to be significant.

TABLE No. 1.

WATERTOWN EXPERIMENTS—CLARK, REEVES & Co. PHŒNIX COLUMNS, FLAT ENDS, WROUGHT IRON.

No	Length. Feet.	Area. Square inches.	r inches.	$\frac{l}{r}$.	Experiment $\frac{P}{A}$ = pounds per square inch.	Formula $\frac{P}{A}$ = pounds per square inch.
1	28	12.062	3	112	35 150	34 445
2	28	12.181	3	112	34 150	
3	25	12.233	3	100	35 270	35 129
4	25	12.100	3	100	35 040	
5	22	12.371	3	88	35 570	35 739
6	22	12.311	3	88	34 360	
7	19	12.023	3	76	35 365	36 480
8	19	12.087	3	76	36 900	
9	16	12.000	3	64	36 580	37 620
10	16	12.000	3	64	36 580	
11	13	12.185	3	52	36 857	39 180
12	13	12.069	3	52	37 200	
13	10	12.248	3	40	36 480	41 496
14	10	12.339	3	40	36 397	
15	7	12.265	3	28	38 157	42 636
16	7	11.962	3	28	43 300	
17	4	12.081	3	16	49 500	49 128
18	4	12.119	3	16	51 240	
19	8 inches.	11.903	3	2.7	57 130	55 495
20	8 inches.	11.903	3	2.7	57 300	
21	25′ 2.65″	18.300	4.4	68.8	36 010	37 078
22	8′ 9″	18.300	4.4	24	42 180	46 056

In Plate XXII, Fig. 1, we see that the proposed formula coincides closely with experiment for small values and large values of $\frac{l}{r}$, but deviates most for $\frac{l}{r} = 40$.

This is precisely what might be expected, because for small values of $\frac{l}{r}$ the flat ends have full bearing. For larger values of $\frac{l}{r}$ the flat ends leave their bearing more or less, while for still larger values of $\frac{l}{r}$, points of contra-flexure become established and the flat ends come back to full bearing again. The deviations shown in Plate XXII, Fig. 1, are thus of just the character which might be expected from a formula which accurately expresses the law of breaking strength.

Colonel Laidley's Experiments.—In Ex. Doc. No. 12, 47 Cong., 1st Session, are given a number of experiments by Colonel T. T. S. Laidley, U. S. A., upon channels with flat ends and columns with pin ends composed of two channels latticed. In Table No. 2, the results of the experiments upon flat end channels, as quoted by Prof. William H. Burr ("Elasticity and Resistance of the Materials of Engineering," New York, John Wiley & Sons, 1883) are given.

The channels were 6, 8, 10 and 12 inches. We have, as before, $n = \frac{5}{2}\pi^2$ from (3). The value of (e) as given by (12) is about $e = 0.8$. The values of μ and E are taken at $\mu = 54\,000$ and $E = 22\,000\,000$ pounds per square inch. We have then $\sqrt{\frac{3\ nE}{\mu}} = 174$ and from equation (11) for $\frac{l}{r} = 0$ to $\frac{l}{r} = 116$

$$\frac{P}{A} = 54\,000\left[1 - \frac{l}{260\,r} - \frac{l}{116\,r}\left(1 - \frac{l}{116\,r}\right)^2\right]$$

TABLE No. 2.

LAIDLEY'S EXPERIMENTS—CHANNELS, FLAT ENDS—WROUGHT IRON.

No.	[Inches.	Area. Square inches	Length. Inches.	r inches.	$\frac{l}{r}$.	Experiment $\frac{P}{A}$ = pounds per square inch.	Formula $\frac{P}{A}$ = pounds per square inch.
1	6	2.33	6.00	0.58	10.3	42 293	47 871
2	6	2.33	17.60	0.58	30.3	36 835	40 014
3	6	2.33	23.90	0.58	41.1	33 910	37 530
4	6	2 33	48.00	0.58	82.6	28 140	33 642
5	8	3.80	8.00	0.48	16.6	43 295	45 068
6	8	3.80	17.90	0.48	37.2	35 280	38 340
7	8	3.80	23.90	0.48	49.7	35 975	36 180
8	8	3.80	29.90	0.48	62.2	33 400	34 884
9	8	3.80	48.00	0.48	99.8	30 620	32 400
10	10	4.85	10.00	0.69	14.5	35 080	45 824
11	10	4.85	17.90	0.69	26.0	33 820	41 320
12	10	4.85	23.90	0.69	34.7	34 355	38 880
13	10	4.85	29.90	0.69	43.4	34 050	37 098
14	10	4.85	48.00	0.69	69.6	34 080	34 365
15	12	6.00	12.00	0.87	13.8	37 240	46 153
16	12	6.00	17.80	0.87	20.5	36 590	43 281
17	12	6.00	23.90	0.87	27.5	36 695	40 845
18	12	6.00	29.90	0.87	34.4	35 150	38 934
19	12	6.00	48.00	0.87	55.2	36 040	35 532

These experiments are plotted in Plate XXII, Fig. 2. It will be observed that there are many anomalous experiments and that the deviations of the formula from the general curve of the experiments for medium values of $\frac{l}{r}$, which have been already alluded to in connection with

Plate XXII, Fig. 1, are still more marked. This again might have been expected. The flat end bearings are for channels very imperfect. It may, indeed, be doubted whether such a cross-section can fairly be considered as flat bearing at all. The case is more probably intermediate between flat ends and pin ends.

Here, however, we are enabled to test the importance of this action of the end bearings, for medium ratios of $\frac{l}{r}$.

Colonel Laidley experimented upon pin-ended columns made of these same channels, latticed. For pin ends there is no disturbing action, such as is due to flat ends, and we should expect the formula to agree more closely with experimental results.

We must take, of course, the same values for μ and E as before, viz., $\mu = 54\,000$ and $E = 22\,000\,000$ pounds per square inch.

The pins were $3\frac{1}{2}$ inches and the channels spaced 8 inches apart. We have therefore from (5) $n = \frac{5}{3} \times 1.2 \times \pi^2$. We have also from (12) $e = 1$ nearly. Hence $\sqrt{\frac{3\,nE}{\mu}} = 155$ and from (11)

$$\frac{P}{A} = 54\,000 \left[1 - \frac{l}{232r} - \frac{l}{116r} \left(1 - \frac{l}{116r} \right)^2 \right]$$

These experiments are given in Table No. 3 and Plate XXII, Fig. 3.

TABLE No. 3.

LAIDLEY'S EXPERIMENTS—PIN ENDS, CHANNELS, WROUGHT IRON.

No.	[Inches.	Area. Square inches.	Length. Inches.	r inches.	$\frac{l}{r}$.	Experiment $\frac{P}{A}$ pounds per square inch.	Formula $\frac{P}{A}$ pounds per square inch.
1	8	7.65	160	3.00	53.3	35 025	34 344
2	10	9.70	200	3.65	54.8	33 920	34 182
3	6	4.65	144	2.35	61.3	34 450	33 388
4	6	4.65	150	2.35	63.9	34 130	33 129
5	8	7.65	200	3.00	66.7	33 790	32 875
6	10	9.70	250	3.65	68.5	33 770	32 713
7	6	4.65	180	2.35	76.7	34 180	32 103
8	8	7.65	240	3.00	80.0	32 375	31 789
9	12	12.00	360	4.44	81.0	31 475	31 719
10	10	9.70	300	3.65	82.2	33 015	31 622
11	6	4.65	210	2.35	89.5	31 935	30 996
12	8	7.65	280	3.00	93.3	31 890	30 623
13	10	9.70	350	3.65	95.9	30 780	30 315
14	6	4.65	240	2.35	102.2	30 085	29 543
15	8	7.65	320	3.00	106.7	29 600	28 846
16	6	4.65	270	2.35	115.0	30 820	27 226
17	8	7.65	360	3.00	120.0	25 885	26 071
18	6	4 65	300	2.35	127.8	24 355	24 256
19	6	4.65	330	2.35	140.6	21 330	21 276
20	6	4.65	360	2.35	153.4	15 320	18 295

TABLE No. 4.

TUBES.—FLAT ENDS, WROUGHT IRON.

No.	Length. Inches.	External Diameter. Inches.	Thickness. Inches.	Area. Square inches.	$\frac{l}{r}$	Experiment $\frac{P}{A}$ = pounds per square inch.	Formula $\frac{P}{A}$ = pounds per square inch.
1	120	1.50	0.10	0.44	240	14 670	13 566
2	120	2.00	0.10	0.61	179	23 206	24 624
3	120	2.35	0.23	1.50	160	21 900	27 573
4	120	2.50	0.11	0.80	141	29 800	29 976
5	120	3.00	0.15	1.35	120	27 670	31 692
6	60	1.50	0.10	0.44	120	31 180	
7	90	3.04	0.17	1.41	90	29 790	33 937
8	60	2.00	0.10	0.61	89	33 300	34 029
9	120	4.05	0.16	1.90	87	26 960	34 200
10	60	2.35	0.22	1.47	80	29 330	34 941
11	60	2.34	0.21	1.37	80	30 000	
12	60	2.50	0.11	0.80	71	35 100	36 041
13	89	4.00	0.24	2.87	67	26 800	36 605
14	90	4.05	0.12	1.61	65	33 330	36 907
15	30	1.50	0.10	0.44	60	34 220	37 722
16	60	4.00	0.24	2.85	45	32 200	40 732
17	30	2.00	0.10	0.61	45	36 980	
18	30	2.35	0.24	1.60	40	35 660	41 952
19	30	2.34	0.21	1.44	40	36 000	
20	29	2.37	0.23	1.55	39	36 910	42 219
21	29	2.34	0.20	1.36	39	39 570	
22	30	2.50	0.11	0.80	35	36 490	43 320
23	28	3.00	0.15	1.41	28	37 390	45 440
24	28	4.00	0.25	2.85	21	48 200	47 817

TABLE No. 4—(*Continued*).

SOLID RECTANGULAR.—FLAT ENDS, WROUGHT IRON.

No.	Length. Inches.	Section. Inches.	Area. Square inches.	$\frac{l}{r}$.	Experiment $\frac{P}{A}$ = pounds per square inch.	Formula $\frac{P}{A}$ = pounds per square inch.
1	120	2.98 x 0.5	1.50	822	8 160	1 240
2	90	2.98 x 0.5	1.50	643	2 410	2 026
3	120	3.01 x 0.77	2.31	540	3 380	2 874
4	120	3.00 x 1.00	3.00	414	4 280	4 888
5	60	2.98 x 0.5	1.50	400	5 630	5 240
6	90	{5.86 x 0.99 / 3.00 x 1.00}	3.00	311	9 600	8 663
7	90	1.02 x 1.03	1.05	300	9 750	9 310
8	120	3.00 x 1.51	4.53	272	10 170	11 325
9	60	3.01 x 0.77	2.31	270	12 970	11 500
10	60	3.01 x 0.99	2.99	207	18 070	19 550
11	60	5.84 x 1.00	5.84	207	17 700	
12	30	2.99 x 0.50	1.50	206	16 850	19 722
13	90	3.00 x 1.53	4.59	204	19 990	20 086
14	60	1.03 x 1.02	1.05	200	17 270	20 805
15	30	3.01 x 0.76	2.30	135	27 770	31 617
16	30	3.00 x 1.00	3.00	104	29 660	33 886
17	30	1.02 x 1.02	1.04	100	25 330	34 100
18	15	1.02 x 1.02	1.04	50	34 550	39 540
19	7.5	1.02 x 1.02	1 04	25	48 680	46 067
20	3.8	1.02 x 1.02	1.04	13	50 400*	50 684

* Bore this without failure.

It will be observed that, so far as the experiments extend, the formula follows the experiments very closely.

For values of $\frac{l}{r}$ greater than 116 and less than 155 we have the straight line,

$$\frac{P}{A} = 54\,000\left[1 - \frac{l}{232r}\right]$$

These experiments seem to confirm our value for n for pin ends, given in equation (5).

In Table No. 4 are given the results of some English experiments upon small flat-end wrought-iron columns of rectangular and tubular cross-section, as given by Prof. William H. Burr ("Elasticity and Resistance of the Materials of Engineering," John Wiley & Sons, 1883). We take for μ and E the same values as in Table No. 1, viz., $\mu = 57\,000$ and $E = 33\,960\,000$ pounds per square inch. We have seen that e for rectangular cross-sections is $e = 1$, and for hollow round, almost the same. Taking then $e = 1$ for both, we have $\sqrt{\frac{3nE}{\mu}} = 210$ and

for $\frac{l}{r} = 0$ to $\frac{l}{r} = 157$, from (11)

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{315r} - \frac{l}{157r}\left(1 - \frac{l}{157r}\right)^2\right]$$

for $\frac{l}{r} = 157$ to $\frac{l}{r} = 210$

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{315r}\right]$$

for $\frac{l}{r} > 210$ $\frac{P}{A} = 837\,900\,000\ \frac{r^2}{l^2}$.

These results are given in Plate XXII, Fig. 4. Here the action already alluded to in the case of flat ends seems very marked. The formula follows, however, quite closely the greater values, and gives, as it should, values a little larger than the greatest. There are many anomalous experimental values, due probably to imperfect centering, etc. The range of the experiment is sufficient to check our assumed values of μ and E. It will be seen that above $\frac{l}{r} = 210$, Euler's curve agrees well with experiment if our value of n, equation (3), is inserted for flat ends.

In Senate Doc. for 1883–84, 1st Session, a number of experiments

are given, made at Watertown upon bars 3 x 3 inches, with 1½-inch pins. The results of these experiments are found in Table No. 5. The coefficient of elasticity for tension is given at about 28 000 000. The tensile breaking stress is given at 50 000. We shall take then $E = 28\,000\,000$ and $\mu = 45\,000$ pounds per square inch.

TABLE No. 5.

WATERTOWN EXPERIMENTS. WROUGHT-IRON BARS, 3 x 3 INCHES, PIN ENDS. PIN 1½ INCHES.

No.	Length. Inches.	$\frac{l}{r}$.	Experiment $\frac{P}{A}$ = pounds per square inch.	Formula $\frac{P}{A}$ = pounds per square inch.
1	30	34.6	28 260	33 511
2	30	34.6	31 990	
3	42	48.5	26 050	30 915
4	42	48.5	30 270	
5	54	62.35	26 320	29 160
6	54	62.35	26 710	
7	60	69.3	26 310	28 467
8	60	69.3	26 640	
9	66	76.2	25 550	27 945
10	66	76.2	25 310	
11	72	83.1	27 930	27 500
12	72	83.1	26 560	
13	78	90.0	26 800	27 085
14	84	97	25 770	26 716
15	84	97	23 490	
16	90	104	24 030	26 370
17	90	104	25 380	
18	95.54	110	25 860	26 032
19	95.54	110	24 240	
20	101.7	117	24 690	25 560
21	101.7	117	22 040	
22	107.57	124	21 870	25 087
23	107.57	124	20 960	
24	113.63	131	19 740	24 500
25	113.63	131	21 050	
26	119.60	137	20 660	23 850
27	119.60	137	20 200	
28	125.64	145	17 580	22 815
29	125.64	145	20 590	
30	131.70	152	20 830	21 735
31	131.70	152	19 470	
32	137.65	161	17 420	20 250
33	137.65	161	18 140	
34	143.82	166	17 490	19 620
35	143.82	166	17 710	
36	149.79	173	16 520	18 450
37	149.79	173	17 840	
38	155.76	180	18 390	17 460
39	155.76	180	16 950	
40	161.81	187	16 590	16 380
41	161.81	187	16 860	
42	167.76	193	15 880	15 480
43	167.76	193	17 920	
44	173.60	200	14 070	14 394
45	173.60	200	14 960	
46	179.50	207	13 010	13 436
47	179.50	207	15 700	

Since from (12) we have $e = 1$, and from (5), $n = \frac{5}{3} \times 1.25\ \pi^2$, we have $\sqrt{\frac{3\ nE}{\mu}} = 196$, and from (11)

for $\frac{l}{r} = 0$ to $\frac{l}{r} = 147$

$$\frac{P}{A} = 45\,000 \left[1 - \frac{l}{294r} - \frac{l}{147r}\left(1 - \frac{l}{147r}\right)^2\right]$$

for $\frac{l}{r} = 147$ to $\frac{l}{r} = 196$

$$\frac{P}{A} = 45\,000 \left[1 - \frac{l}{294r}\right]$$

for $\frac{l}{r} > 196$

$$\frac{P}{A} = 575\,750\,000 \frac{r^2}{l^2}.$$

These experiments are given in Plate XXII, Fig. 5, together with the straight line formula, and the Rankine-Gordon formula,

$$\frac{P}{A} = \frac{40\,000}{1 + \frac{l^2}{18\,000r^2}}$$

The experiments show many anomalous results, as will be seen from the values in Table V, for Nos. 1 and 2, 3 and 4, etc. Also the experiments give a less breaking load for $\frac{l}{r} = 145$ than for $\frac{l}{r} = 180$. Such discrepancies must be attributed to lack of centering, etc. The formula follows the general law well when tested by maximum values.

Concluding Remarks.—The proposed formula has a rational basis, and depends only upon the physical properties of the material, the cross-section and the end conditions. No single formula hitherto proposed will embrace all the experiments here given so well. Still, more comparisons are needed in order to thoroughly establish the reliability and accuracy of the formula. It is for that reason that the writer calls the attention of the Society to it. He has simply endeavored to show by the present paper that there seems to be a strong presumption in its favor and that it is worth investigating. In order to thoroughly test it, careful experiments are needed over a wide range and various conditions, for which the values of μ and E are known by actual experiment in each case.

It is to be regretted that in all the experiments found available, the physical properties of the material experimented upon are

not determined, and must be estimated. It is this fact which weakens the force of the present comparison. The values of μ and E are not known certainly, and the suspicion must arise that in assuming them the writer has really fitted his formula to the experiments in each case.

A scrutiny of the plates will remove this objection. In nearly all cases, the average results could be improved by taking different values of μ and E. In Plate XXII, Fig. 5, we fortunately know μ and E very nearly. In Plate XXII, Fig. 4, the agreement with Euler's curve for large values of $\frac{l}{r}$ sanctions our value for E, while the range of experiments is such that μ can be closely assumed. In Plate XXII, Fig. 1, we have taken the same μ and E as in Plate XXII, Fig. 4. The variations of the formula from experiment are in the main what we should expect from an accurate formula when compared with experiment.

In any case it seems beyond doubt that even with average values of μ and E, equation (11) is preferable to either the straight line formula or the Rankine-Gordon formulas in general use.

Christie's Experiments.—In the *Transactions* Am. Soc. C. E. for April, 1884, Mr. James Christie, M. Am. Soc. C. E., has given the results of about three hundred experiments upon wrought-iron struts, embracing angles, tees, beams and channels, and tubes, for flat ends, fixed ends, pin or hinged ends, and round ends.

The value of E for compression, appears from experiments by Mr. Christie to be about 24 000 000 pounds per square inch. The experiments, when plotted, indicate the breaking stress μ to be about 57 000 pounds per square inch.

For flat ends we have, then, $n = 24.675$, and $\sqrt{\frac{3\,nE}{\mu}} = 176$. If we take $e = 1$, we have

Flat Ends.—

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{264r} - \frac{l}{132r}\left(1 - \frac{l}{132r}\right)^2\right] \text{ for } \frac{l}{r} = 0 \text{ to } \frac{l}{r} = 132$$

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{264r}\right] \text{ for } \frac{l}{r} = 132 \text{ to } \frac{l}{r} = 176$$

$$\frac{P}{A} = \frac{nEr^2}{l^2} = 592\,200\,000\,\frac{r^2}{l^2} \text{ for } \frac{l}{r} > 176.$$

For pin ends, taking $n = \frac{5}{3}\pi^2 = 16.45$, $e = 1$, and same values of E and μ as before, we have $\sqrt{\frac{3\,nE}{\mu}} = 144$ and

Pin Ends.—

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{216r} - \frac{l}{108r}\left(1 - \frac{l}{108r}\right)^2\right] \text{ for } \frac{l}{r} = 0 \text{ to } \frac{l}{r} = 108$$

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{216r}\right] \text{ for } \frac{l}{r} = 108 \text{ to } \frac{l}{r} = 144$$

$$\frac{P}{A} = 394\,800\,000\,\frac{r^2}{l^2} \text{ for } \frac{l}{r} > 144.$$

For round ends we have $n = \pi^2 = 9.87$, and taking $e = 1$, and same values of E and μ as before, we have $\sqrt{\frac{3\,nE}{\mu}} = 112$ and

Round Ends.—

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{168r} - \frac{l}{84r}\left(1 - \frac{l}{84r}\right)^2\right] \text{ for } \frac{l}{r} = 0 \text{ to } \frac{l}{r} = 84$$

$$\frac{P}{A} = 57\,000\left[1 - \frac{l}{168r}\right] \text{ for } \frac{l}{r} = 84 \text{ to } \frac{l}{r} = 112$$

$$\frac{P}{A} = 236\,880\,000\,\frac{r^2}{l^2} \text{ for } \frac{l}{r} > 112.$$

TABLE No. 6.

CHRISTIE'S EXPERIMENTS—AVERAGE RESULTS.

WROUGHT-IRON ANGLES, TEES, CHANNELS, BEAMS AND TUBES.

$\frac{l}{r}$	Flat ends $\frac{P}{A}$ = pounds per square inch.	Flat ends formula $\frac{P}{A}$	Pin ends $\frac{P}{A}$ = pounds per square inch.	Pin ends formula $\frac{P}{A}$	Round ends $\frac{P}{A}$ = pounds per square inch.	Round ends formula $\frac{P}{A}$
20.....	46 000	46 466	46 000	44 716	44 000	42 408
40.....	40 000	39 974	40 000	38 076	36 500	36 024
60.....	36 000	36 337	36 000	34 912	30 500	33 345
80.....	32 000	34 371	31 500	33 060	25 000	29 754
100.....	29 800	32 889	28 000	30 324	20 500	23 085
120.....	26 300	30 666	24 300	25 308	16 500	16 450
140.....	23 500	26 733	21 000	20 064	12 800	12 084
160.....	20 000	22 458	16 500	15 422	9 500	9 253
180.....	16 800	18 280	12 800	12 185	7 500	7 311
200.....	14 500	14 805	10 800	9 870	6 000	5 922
220.....	12 700	12 240	8 800	8 160	5 000	4 900
240.....	11 200	10 280	7 500	6 854	4 300	4 112
260.....	9 800	8 760	6 500	5 840	3 800	3 503
280.....	8 500	7 554	5 700	5 023	3 200	3 012
300.....	7 200	6 580	5 000	4 387	2 800	2 743
320.....	6 000	5 783	4 500	3 856	2 500	2 313
340.....	5 100	5 123	4 000	3 415	2 100	2 050
360.....	4 300	4 570	3 500	3 046	1 900	1 828
380.....	3 500	4 100	3 000	2 734	1 700	1 640
400.....	3 000	3 700	2 500	2 468	1 500	1 480
420.....	2 500	3 357	2 300	2 240	1 300	1 343
440.....	2 200	3 059	2 100	2 040	.....	.....
460.....	2 000	2 710	1 900	1 866	.....	.....
480.....	1 900	2 570	1 800	1 713	.....	.....

In Table No. 6 can be found the results of these formulas compared with the combined average values of all the tests, as given by Mr. Christie, and in Plate XXII, Fig. 6, the flat-end experiments of Mr. Christie are shown. Experiments upon angles are denoted by (r), upon tees by a black dot (.), upon channels and beams by (x), and upon tubes by (o). The formula curve has been drawn full, and the "straight line formula" broken.

It will be observed that although the formula results vary somewhat from the average results of Table VI, its curve accords well with the actual experiments, if anomalous ones are omitted.

It will be also observed from Table VI that the formula agrees equally well with experiment in all three cases of flat ends, pin ends and round ends.

DISCUSSION.

THOS. H. JOHNSON, M. Am. Soc. C. E.—In the course of this paper, Prof. DuBois has made frequent reference to the paper on "Strength of Columns," which I had the honor to present to the Society at the Deer Park Convention, in 1885. In some instances his present paper agrees with my own, and in others it is at variance. On page 523, Vol. XV, *Transactions* of this Society, I had shown that the coefficient in Euler's equation, to agree with the experiments, should be taken at for round ends, π^2; for hinged ends, $\frac{4}{5}\pi^2$; for flat ends, $\frac{4}{2}\pi^2$. The fractions $\frac{4}{5}$ and $\frac{4}{2}$ were obtained by the purely empirical method of comparing, one with another, the lines obtained from the plotted experiments, in the three forms of end bearings.

In the paper now before us, Professor DuBois has endeavored to deduce these relations by analytical methods, and has arrived at the same constants, which had been previously found by the other method. This confirmation is gratifying, and would be more so, if I were able to follow the Professor's reasoning more closely. But I am free to confess that I do not quite see how his equation (4) follows from the premises previously stated. His equation (5) reduces to $\frac{4}{2}\pi^2$, only when $d=0$, which is to say, when the pin is reduced to a knife-edge; whereas, the plotted experiments show that the value of the constant applies when the pins have the proportions of ordinary practice.

It seems to me that, if the pin is reduced to a knife-edge, the frictional resistance will be eliminated, and the resistance to flexure, in a plane at right angles to the pin, will be the same as for round ends.

Hence, the formula, which takes cognizance of the diameter of the pin, ought to be of such form that the constant will become π^2 when $d = O$. Whether it should become $\frac{5}{2}\pi^2$, the same as for square ends, when $d = D$, as Prof. DuBois assumes, is not clearly apparent. The square end presents a material obstacle to change of direction at the end, in the fact that the diagonal dimension of the column is always longer than the side, while a pin resists such change of direction only by the friction on its surface. For a given allowed unit strain in bearing, we would have equal areas of bearing surface, for equal loads, whether the pin were large or small; and with equal loads on equal surfaces the amount of friction will be the same. The only difference, as between large and small pins, lies in the fact that the friction acting at the surface of the pin will have a longer lever arm in the case of the larger pin. Whether this resistance will equal the square end resistance, when the diameter of the pin is equal to the side of the column, remains to be proven, and cannot be assumed out of hand. It is more probable that if such equality of resistance occurs at all, it will be found at some fixed ratio of d to $\frac{l}{r}$ instead of a fixed ratio of d to D, because the bending moment, which calls into play the end resistance, is a function of $\frac{l}{r}$ and not of D.

The most notable difference between Professor DuBois' treatment of the subject and my own, lies in this: that I consolidated all the available experiments into one group, and sought a formula that best expressed the average results of all the tests, except a few abnormal ones; but Prof. DuBois has rejected the great mass of tests, and sought an expression that would represent only the few tests that give maximum results, including those exceptional ones which should be rejected as outside the action of the general law, because of exceptional conditions. Why he should thus exalt the few and reject the many he fails to state. On Sheet No. 41, of Vol. XV, 355 experiments in square end columns are plotted, of which nineteen are outside the field of the formula therein developed. Of these, thirteen are near the point B, and in Prof. DuBois' Fig. 5 six are near his point D. In addition to these, about twenty-one others give maximum results at other points. According to the method followed in the paper now before us, Prof. DuBois would have given undue weight to the forty maximum results, nineteen of which are accidental and abnormal, and would have practically ignored the remaining 315 experiments.

In order to more fully understand the meaning of these irregular results of actual tests, let us consider briefly the underlying principles so far as they are accessible.

Euler's equation is derived from the equation of the elastic line, and is based on the assumption that the entire energy of the imposed

load is expended in producing a bending moment in the column, which is resisted by the elastic reaction of the column considered as a bent spring; and no part of the load is transmitted to the base by direct compression. This equation as applied to very long columns has been mathematically demonstrated; is sanctioned by such authorities as Weisbach, Rankine and others; is abundantly confirmed by experiment, and must be accepted as fully established for very long columns.

On the other hand, when the column is very short, it is equally well established that failure occurs by direct compression without flexure. Between these extremes failure occurs partly by compression, and partly by flexure; the former predominating in the shorter length-ratios, and the latter predominating in the longer length-ratios, as we approach the line of Euler's curve, when the direct compression wholly disappears. It may happen that perfect homogeneity of material and exact fitting in the testing machine may prevent flexure, and lead to results due wholly to compression, even with specimens having very considerable length. In illustration of this point, permit me to refer to *Transactions*, Am. Soc. C. E., Vol. XIII, pages 96 to 111, inclusive, wherein will be found no less than seventeen cases in which Mr. James Christie reports "No Failure at 50 000 pounds." These cases embrace five tests in which $\frac{l}{r}$ is between 50 and 100; ten cases with $\frac{l}{r}$ between 100 and 200, and two cases with this ratio over 200. The causes leading to these results are discussed on page 524 of Vol. XV of the *Transactions*.

With such results before us, it is evident that failure by compression alone without flexure may be of frequent occurrence in the shorter lengths, and in fact nearly every group of tests, of which the writer has knowledge, shows evidence of such results between lengths of $\frac{l}{r}=0$ and $\frac{l}{r}=30$. But such results whenever recognized must be regarded as tests of compression resistance only, and in seeking for the law governing columns under mixed strains of compression and flexure, such tests should be rejected as accidental and not within the pale of the law sought. On the other hand, near the other limit, if the material has a low value of E, or if through internal strains, the column is predisposed to flexure, we will have the case of failure by flexure alone, when the length-ratio should give mixed strains. Of this character are the few experiments on Plates 41 and 42, Vol. XV, which lie above the line of maximum limit. In such cases the result should approximate to the curve of Euler's equation, and a glance at the plates shows this to be the fact.

In dealing with each group of experiments separately, the limited number of experiments gives greater relative value to all irregularities,

caused by variation in the physical properties of the material, inaccuracies of workmanship, etc., and thus the governing law may be obscured. By combining all the available experiments into one group, the irregularities of one group are balanced by contrary irregularities of another group, so that the greater the number of experiments, the greater the probability of arriving at the truth. The method of dealing with each group separately is apt to be misleading and has, in fact, misled Professor DuBois in this paper, as shown by the fact that, taking the six groups of experiments illustrated by him in Plate XXII, Figs. 1 to 6, it will be seen that with the exception of Figs. 1 and 3, the line representing my formula is a better exponent of the experiments than that which he now proposes.

When Professor DuBois, in the course of his article, is pointing out in detail the specific points wherein his new formula differs from mine, he might have added that my formula represents the average of a great many experiments; his a few only, and those the exceptional and erratic ones. This one difference is primary and fundamental, and covers and includes all the others. I am content to let the profession at large judge between us as to which has followed the rational and proper treatment.

The comments following equation (10) are misleading and need some attention. When he states that his equation differs from mine in that "its point of tangency and its inclination depend on the actual value of μ in any case," he leaves the inference that my formula did not depend on the known physical properties of the material. A reading of my original paper will dispel any illusions on that subject. On page 524, Vol. XV, of the *Transactions*, attention is called to the fact as a prime feature of the formula, that it is readily applicable to any material of which the moduli of compression and elasticity are known. This feature is, of course, broad enough to cover all variations in these values occurring in different groups of experiments on the same material.

Again, he says that all experiments which he has been able to plot "show from B to D, a curve similar to that shown in Fig. 5." This is equivalent to saying that every group of experiments shows some signs of internal strains near D, and that very short lengths show a tendency to fail without flexure. The latter point has already been conceded, and the Professor's diagrams show that these results are confined to values of $\frac{l}{r}$ less than 20. To better understand what this $\frac{l}{r}=20$ means, it may be stated that this ratio corresponds to a length of 5.8 diameters for a solid square and 5 diameters for L, T, and $+$ shapes. It is needless to say that flexure in such short pieces is scarcely possible. In regard to the point D, it is rather straining the facts to claim that his Plate XXII, Figs. 2 and 6, show any marked tendency of the

tests to give high results at that point. Of the plates which accompany my paper, in Vol. XV, Nos. 41 and 42, as already stated, show high results near "*D*," the accidental character of which has already been discussed. But in the remaining plates this feature is wholly wanting, and they include the following classes:

Plate 43, Iron—round ends.

Plates 44 and 45, Mr. Christie's experiments on mild and hard steel, respectively.

Plates 46 and 47, Mr. Hodgkinson's experiments on cast iron, flat and round ends.

Plate 48, oak, flat ends.

It is to be regretted that Professor DuBois overlooked such well-known tests as those of Christie, on steel, and Hodgkinson, on cast iron; especially as they would have led him to modify his formula.

He further objects to my making OB^1 (K of my formula) less than his μ, although he concedes that it strikes an average of the experiments. He says that it follows no given set of experiments. His own diagrams refute that statement, as well as those which accompany my own paper.

Near the close of the paper, the professor naively remarks that the "curve" (of his formula) "accords very well with the actual experiments, *if anomalous ones are omitted.*" (Italics are mine.) In view of this remark, it is interesting to note how many "*anomalous*" experiments are to be found in Plate XXII, Figs. 4 and 6, of his paper, and 41 and 42 of mine.

In *Engineering News* of December 22d, 1888, and February 23d, 1889, I took occasion to point out the error in Tredgold's hypothesis, which is the fundamental basis of the Gordon–Rankine formulas, and all others of that form. Tredgold assumed that the imposed load produces a direct compression equal to the full amount of said load; and that it produced a bending moment, also, equal to the full amount due to said load. In other words, that a given amount of force may develop double the amount of energy due to that force, which is contrary to a well-established principle in mechanics. It was also pointed out that the true hypothesis must recognize the fact that a given amount of force is capable of developing only a definite and fixed amount of energy. If a part of that force is expended in producing flexure in the column, the unexpended residue alone can be expended in producing direct compression. The accidental results above referred to are in accord with this hypothesis, and furnish the "exceptions which prove the rule"; but they are none the less exceptions, so far as relates to the law governing columns under conditions of mixed strains.

According to views expressed in the discussion of Professor Waddell's

recent paper, it would appear that there are some members of the profession who still regard the Gordon-Rankine formula as the only correct one, and that the straight-line formula is a "fad," a "labor-saving approximation," etc., and Professor Swain goes considerably out of his way to demonstrate that the straight-line formula cannot be true. But he falls into the remarkable error of assuming that this formula is an expression for the "fiber strain," which it is not. All the formulas which have been heretofore proposed, Hodgkinson's, Gordon's, Rankine's, Cooper's and others, including the straight-line formula, are expressions for the imposed load, and not for the fiber strain produced by that load. Failure occurs when the fiber strain reaches the limit of endurance of the material. That limit of endurance is produced in a long column by a smaller load than in a short one. These loads have been observed in experimental tests, and the formulas deduced therefrom are expressions for the imposed load. But no attempt has been made as yet to determine a formula for the fiber stress due to a given load. If such minds as those of Professors DuBois, Swain, and others that might be named, would turn their attention to elucidating a formula from the hypothesis just pointed out, and can demonstrate in what proportion the energy of the load is divided between flexure and compression under the different conditions of length and end bearing, they will probably enable us to compute the fiber strains, and, in addition to that, will provide us with the means of determining the proper size of lacing bars on open sections, and the number of rivets to each—problems in practical work which are, as yet, without solution.

Those other gentlemen, whose regard for the great name of the illustrious Rankine will not permit them to accept any other than the formula which bears his name, and who look upon the straight-line formula as a makeshift and a "fad," have not only failed to study the evidence on which the straight-line formula is based, but have no doubt overlooked that sentence of Professor Rankine's (page 360, *Applied Mechanics*), in which he introduced the aforesaid formula with the positive statement that the "theory" is unsatisfactory, and the formula "must be regarded as provisional only." It will be in order for these gentlemen, before further assailing the later formula, to show wherein that great man erred when he wrote that sentence. Did he not see and appreciate the fallacy of the Tredgold hypothesis on which the formula was based?

HENRY B. SEAMAN, M. Am. Soc. C. E.—A rational formula, based on sound theory and deduced with mathematical accuracy throughout, one of sufficient simplicity to be of practical value, is much desired. By it we would expect to calculate the elastic strength of a column, with the same certainty that we now calculate the elastic strength of a girder.

The most simple column formula yet devised is the straight line, and with any of its numerous constants, it agrees fairly well with the results of experiment between the limits for which it is intended. The objections to its use are that its application is limited, and the adoption of any empirical formula, instead of a rational one, is to be deprecated.

The proposition to adopt three formulas, to serve the purpose of one, each of these three acting between limits, and consequently none of them mathematically correct, could hardly be received with favor; and when we add to this the fact of empirical deduction, any advantage over formulas already in use, disappears altogether.

Professor Du Bois proposes the use of three formulas, each between limits, and claims for them rational origin, and the greatest conformity with the results of experiment.

On page 76 he says: "For the curve from D to B, Fig. 5, the writer has, after much labor and testing, assumed the form $y = \mu\ [1 - ax - bx\ (1 - bx)^2]$." The assumption of this value of y is a purely empirical procedure. He has made several graphical illustrations, showing the accuracy of his combined formula, as compared with the straight line and with the Rankine-Gordon. The most valuable of these plots is that of Christie's experiments, since they are the most complete. He has plotted here, both his own formula and the straight line, but has omitted the Rankine-Gordon. If he will plot the latter for long columns on this plate, it will be found superior to either of the others. For short columns, all agree as well with experiments, as the experiments agree with each other.

The statement in the paper that a point is reached where pure flexure occurs is erroneous. Pure flexure never occurs in a column. The column may be so long that after deflection, the direct strains are slight as compared with those due to flexure, but we may as well ignore the shear in long girders as to neglect the direct compression in a column. When the applied forces are in the same straight line and in opposite directions, there must be direct resistance. Similarly, if the coefficients E for tension and compression are the same, the neutral axis cannot be in the centre of gravity of the section, since the forces of compression exceed those of tension by an amount equal to the applied force.

With these conditions before us, it is difficult to conceive why a column should suddenly "spring" from a condition of combined strains to one of pure flexure, and if such a phenomenon has ever been observed, it is probably the change from the condition of direct compression to one of combined strains.

In applying his formula, the author has varied the values of E over 50 per cent. and μ varies from 45 000 to 57 000 pounds, or over 25 per cent. There would seem to be enough latitude here to bring almost any desired result.

A. J. DuBois, Jun. Am. Soc. C. E.—The true curve for the strength of columns, as indicated by experiments, seems to be one which for large values of $\frac{l}{r}$ coincides closely with Euler's curve, with the values of the coefficient as given by Mr. Johnson and deduced in the present paper. For medium values of $\frac{l}{r}$ the curve becomes approximately a straight line, while for small values of $\frac{l}{r}$ we have again a curve running up to the full value of μ for $l = 0$.

No one has yet deduced the single equation which corresponds to this law. The Rankine-Gordon formula certainly does not. It can be fitted by a proper choice of constants, to a given group of experimental values, generally on the safe side, for those values of $\frac{l}{r}$ which are of practical importance, and it is thus used. But as an expression of the general law it is very defective. I for one fully agree with Mr. Johnson's remarks concerning it.

To Mr. Johnson belongs the credit of using Euler's curve with proper coefficients, in conjunction with a straight line, and of showing that the results thus obtained hold good over a greater range. The construction certainly seems to approach nearer to the true curve.

But while Mr. Johnson places a limit to Euler's curve, he runs his straight line through from this limit to the axis, and ignores the curve for small values of $\frac{l}{r}$. The consequence is that for $l = 0$, he gets a result less than μ!

Practically, as he points out, these small values are of little importance. But if by the introduction of this curve for small values of $\frac{l}{r}$ we obtain different limits for the straight line and also a different straight line, it becomes of considerable importance to consider this curve for small values of $\frac{l}{r}$, not for itself, but for its influence on the rest of the curve.

This is what I have attempted to do in the present paper. Mr. Johnson uses Euler's curve and a straight line. I use Euler's curve, a straight line and another curve, and my straight line has different limits and location. For $l = 0$ I obtain μ and not a result less than μ.

The question is, have I a closer approximation to the true law?

To answer this question I have given diagrams showing the curve of my formulas as compared with experimental results, and the Rankine-Gordon and Johnson formulas.

It seems to me that these diagrams show a closer agreement for the formulas I have given, while Mr. Johnson claims the closer agreement for his own.

He claims that I give undue weight to exceptional results, and thus "exalt the few and reject the many."

I might reply, that if results more in accord with Mr. Johnson's idea of the "average" are desired, they can easily be obtained. If my object had been to thus fit the experiments, I could easily have obtained results much closer to the average than Mr. Johnson's, and over a greater range. I have three curves to adopt while he has but two. But this is not my object. The main point of the paper is that there is no attempt to thus fit the experiments. The constants of my equations and their limits are given by the physical qualities of the material only. I do not take $\frac{2}{3}\mu$ in place of μ, in order to get better average results. I might take this stand.

But my real reply to Mr. Johnson's point is, that I consider maximum results nearer the truth than "average" results. I do not consider that I have given undue weight to "abnormal" results, but I do think that Mr. Johnson gives undue weight to "average" results. I will give my reasons for so thinking. They may be naive, but they seem to me common sense, and I think they will seem so to others.

Perfect homogeneousness of material and perfect fitting, in the testing machine, may result in an abnormally large result, even for a column of considerable length, as Mr. Johnson points out. But it is difficult to secure either one of these conditions and still more difficult to secure them both simultaneously. On the other hand lack of homogeneousness and imperfect fitting are much more common and result in an abnormally small result. Abnormally small results are thus much more liable to occur than abnormally high results, and the true curve should therefore lie considerably above the average. This is just what I obtain from my formulas without doctoring the coefficients.

Moreover, such abnormal results as Mr. Johnson refers to are easily detected. A reference to the diagrams will show such, and will also show that my curve runs under them, and that I do not exalt them nor use them as Mr. Johnson states.

But when a series of maximum results follows in regular sequence and gives a regular curve of the same general law and general form as the average curve, without abnormal breaks, I do not regard these maximum results as "abnormal." On the contrary, I consider them as more correct than the average, for the reasons given.

With this statement, I refer readers to the diagrams and leave them to decide between us, simply stating that the high values given by my formulas which Mr. Johnson considers a defect, I consider as an indication of approximation to the true curve, and therefore as a merit.

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(Vol. XXVII.—July, 1892.)

DISCUSSION ON PAPER No. 529.—"THE MAIN RELIEF SEWER OF BROOKLYN."*

By WILLARD BEAHAN, M. Am. Soc. C. E.

I desire first, in concluding this discussion, to make a few corrections. On the illustration, page 497, the words "Iron poling *bar*" should read "Iron poling *board*." On page 513 the words "*fair* section of tunnel" should read "*full* section of tunnel." On page 517 the closing lines of the first paragraph were not intended for publication.

In reply to Mr. Sears, I wish to state that the method pursued in treating the subject was my own. The paper was written from the standpoint of a tunnel foreman, *construction* rather than design being the central idea, and the intent being to give the results of experience. Weisbach's or Kutter's formulas had little to do with the matter, and Mr. Sears' sarcasm in this connection is out of place.

On plate XLIV, the progress made is given by months. The cost to the city has already been published in full elsewhere. It was not my intent to enter upon this question, and the criticism on this point is therefore uncalled for.

The same reply can be made to the first part of Mr. Nichols' discussion. I wish to say here that the shield system for rapid transit construction under Broadway in New York, was spoken of by me with the plans of Mr. Worthen in my mind. This line is shallow in depth, and generally in drift material.

Mr. Nichols thinks I am "unnecessarily sarcastic" towards engineers. It has been my experience that engineers as a class are in-

* This discussion, which is a correction and an additional reply by the author, was not received in time for publication with the paper.

tolerant, having little *esprit de corps*, and not apt to give due credit to the practical men working with them.

The Hudson River Tunnel is the pioneer of tunnels in soft ground in this country. The histories by Mr. Brush and Mr. McCann are very interesting to me. Mr. Brush, however, will agree with me, I think, that he gave an unfortunate turn to the discussion. The whole question of priority of idea and plan of execution in no way concerns the subject of my paper, and was but incidentally touched upon by me. It is also foreign to the *Transactions* of our Society. As the statements made in the paper have nevertheless been questioned, I think it is but right to say that Mr. Anderson recently told me that he obtained the idea of the pilot while sinking cylinders for the bridge at Omaha, as a foreman under T. E. Sickels, M. Am. Soc. C. E., Chief Engineer. One of the cylinders split in sinking, and they did not think best to continue the use of compressed air. Mr. Sickels devised the plan which was carried into execution, of building it downwards by means of plates with inside flanges.

Members who desire to examine this subject further will find it treated with much detail in Mr. S. V. Burr's work on the Hudson River Tunnel. Much information may be obtained also from records in the newspapers at the time of the tunnel accident. Mr. Anderson has held a patent on the use of the pilot in tunnel work for some ten years.

Referring to Mr. Hutton's remarks about shields, I would say that the shields used in South 5th Street resembled, I believe, the Great-head. The point I would make is that a shield cannot be used in non-flowing material, and I regret that some advocate of the use of shields has not discussed that point, as it excludes shields from 90 per cent. of all tunnels. If one man bids on work intending to use shields, and another on any other workable plan, the latter will always be able to underbid the former.

As to Mr. Hutton's contention that shields are not "pneumatic appliances," very well—where are they most used if not in water or flowing silt? They certainly are not good pneumatic appliances. If, for example, in his work last year, he had carried enough air pressure in the Hudson River Tunnel to insure a dry floor, he would have had some fourteen pounds pressure in excess of the water pressure at the roof, which, in the silt through which the excavation was being made, could not be otherwise than very dangerous. True, he carried less pressure than this, but at the risk of drowning the laborers in front of the shield in case it went out of slit into "cold water."

The Messrs. Pierson, in their work recently undertaken in London, are reported to have divided the shield used into two parts horizontally, so as to enable them to carry more pressure at the floor than at the roof. This is the latest improvement in shields, and a useful one in avoiding "blow-outs."

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(Vol. XXVII.—August, 1892.)

UNIFORM PRACTICE IN PILE-DRIVING.

By J. FOSTER CROWELL, M. Am. Soc. C. E.

WITH DISCUSSION.*

The absence of anything like uniformity in the adaptation of piles to foundations and other structures cannot have failed to attract the attention of many engineers, probably of all who have had occasion to investigate the subject.

The diversities of practice and of individual views seem to be more marked in this than in almost any other ordinary structural proceeding, and there seems to have been no evolution, during the last twenty-five years at least, toward agreement as to the essential requirements. After making due allowance for the personal characteristics of engineers who, on the one hand, through conservatism or timidity, provide too liberally, or, on the other, through boldness or over-confidence venture to the point of insecurity, it will be admitted that there are wide variations, even in what may be termed good practice, where we

* See page 129.

would naturally expect conformity to a standard. This is the more surprising when we consider that the general principles involved are neither obscure nor unrecognized; it is in the practical application of the principles, chiefly, that the differences occur.

The writer having had a somewhat wide range of experience with this detail of construction (embracing pile bridges, trestles and wharves; foundations of bridge piers and abutments, buildings and machinery; covering cases in various situations and soils, including firm earth, treacherous clays, quicksands, deep swamps and sea sand) has often been impressed with the practical importance and economic value of securing more uniform practice, and this end seems still to be growing more desirable, in view of the extensive use to which piles are being put in some localities for foundations of colossal and lofty buildings with enormous weight concentrations, as well as in more ordinary application generally.

The term uniform practice in connection with a subject so diversified may need definition. As used here it is meant to express consistent adherence to a recognized standard in all cases, and, as nearly as possible, parity under similar conditions. With the desire to contribute, in so far as he may be able, to this object, and to evoke discussion which may be of permanent efficacy, the author ventures to offer for the consideration of his fellow-members a few simple observations connected with a comparison of different authorities and formulas in vogue, which he recently had occasion to prepare in his own practice. The nature of the comparison is entirely practical and relates only to authorities that are generally recognized and resorted to.

At the outset it may be well to bear in mind that in order to discuss the subject intelligently, we must keep separate the extreme sustaining power of any pile under a static load, and the character of the particular use or stress to which the pile is to be subjected. We will first consider a series of differing values, obtained by several different recognized formulas, for the sustaining power of the same pile under exactly the same conditions; we will then examine those formulas in detail and endeavor to select the most satisfactory one as a general formula for practical use; and, finally, we will study the consistent application of the formula so selected, to the varying conditions met with in actual practice.

Rudolph Hering, M. Am. Soc. C. E., in a very valuable mono-

graph on Bearing Piles,* which cannot be too highly recommended as a most useful work of reference, has collated and tabulated no fewer than fourteen different formulas attributed to ten different authorities, which give fourteen different values for the extreme sustaining power of the same pile, driven under precisely similar conditions; the values range, in a typical case, all the way from 96 000 to 600 000 pounds, without taking into consideration the further differentiation resulting from varying views as to suitable coefficients of safety, which run anywhere from $\frac{1}{2}$ to $\frac{1}{100}$, so that if we were considering the working load for a given pile under extreme conditions we should find it by one authority to be 4 500 pounds; by another, 16 000; by a third, 48 000; by a fourth, 75 000; by a fifth, 120 000, and so on. It is to be remarked that one of the most eminent of the authorities gives five different formulas of varying theoretical exactitude, two others each give two forms to be used in separate cases which in practice it is quite difficult, even for an expert, to choose between; while none of the authorities attempt, except in the most general way, to make any classification of applications, but leave that entirely to the individual judgment, within very wide margins. This is most perplexing to those whose need of a working formula is most urgent and, in connection with the difficulty of making a correct prognosis, naturally tends to produce the great disparities in practice which we have noted and which it should be our aim to avoid in order to attain to absolute economic security. The list of formulas best known and which represents practically all the valuable literature on the subject, is as follows:

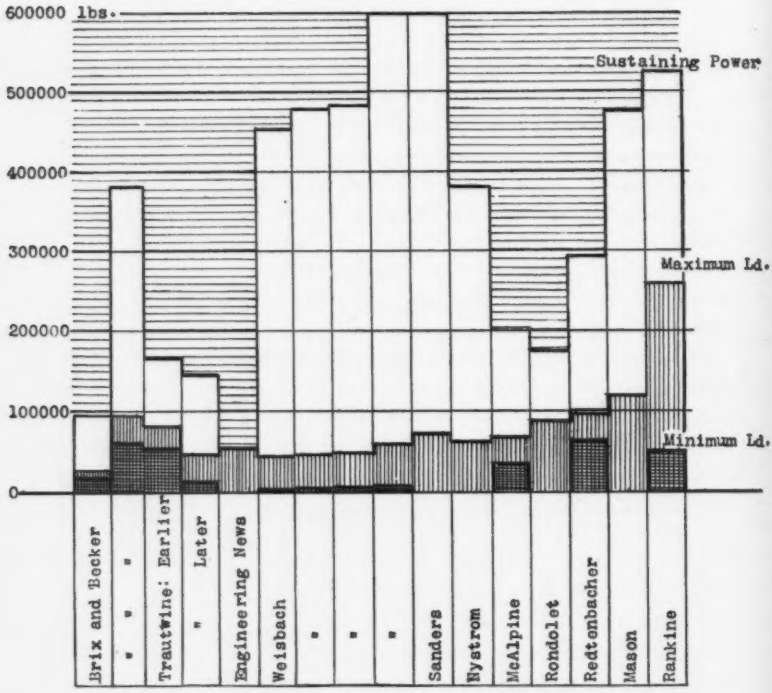
Weisbach.....in five forms.	Redtenbacher.
Sanders....one form of Weisbach.	Brix and Becker ....in two forms.
Mason.....one form of Weisbach.	Nystrom.
Trautwine.....in two forms.	McAlpine.
Rankine.	*Engineering News.*
Rondolet.	

The diagram (No. 1) exhibits the deduced values of extreme sustaining power, and the ordinary and the minimum loads recommended by each authority in the case of a typical pile driven under ordinary conditions. The purpose of this diagram is merely to show variations in results.

* "Bearing Piles, by Rudolph Hering, M. Am. Soc. C. E. Engineering News Publishing Co. New York, 1887."

COMPARISON DIAGRAM No. 1.

SHOWING DIFFERING VALUES BY VARIOUS FORMULAS IN THE CASE OF A 2 000-POUND RAM FALLING 30 FEET, WITH SET OF .1 FOOT (= 1.2 INCHES), OF EXTREME SUSTAINING POWER AND PERMISSIBLE LOADS.



It is evident from inspection of Diagram 1, that the authorities, as has already been stated, differ, not only as to safe loads, wherein there is reasonably room for much divergence, but also to a very marked extent in the theoretical extreme sustaining power. The "factor of safety," too, seems to be entitled to the nickname once bestowed upon its cousin of the iron bridge. But if we look a little closer at the forms of the formulas we shall see why such variations occur, and may perceive that if we had taken some other case, with a different weight of hammer and a different fall, we should have obtained another, but quite different, set of variations, and so for every assumed case.

For our present purpose it will not be necessary to review the entire list of formulas, a few being sufficient for illustration. All the formulas are based, of course, upon the mechanical principle of accumulation of work, which may be stated in the following propositions:

First.—The work accumulated in a body, whose motion is accelerated through any given space by a given force, is equal to the work which it would be necessary to do upon the body, to cause it to move back again through the same space when acted upon by the same force, or through a proportionate space, when acted upon by a different force.

Second.—The resistance offered to a moving body in which work has been accumulated, is inversely proportional to the space through which the body moves before coming to a state of rest, after the resistance is applied.

In the case of the pile-driver the accumulated work in the falling hammer or ram is met by the resistance of the pile and its support, and the space through which the ram moves downward after reaching the pile, represents inversely the measure of the resistance. So far it is a mere question of dynamics, and if we could eliminate the elasticity and compressibility of hammer, pile and soil; neglect the consideration of their relative weights, and the frictional and other resistances to the motion of the hammer; then the theoretical resistance, or sustaining power of the pile would be found in the simple expression:

$$\text{Sustaining power} = \frac{\text{Weight of ram} \times \text{fall}}{\text{Space through which the hammer moves after reaching the pile.}}$$

But we cannot, as a matter of course, treat the matter thus simply, for it is in reality a very complex case of impact, wherein the exact application of principles to any particular case is well-nigh impossible under ordinary working conditions, where the falling ram is used; but which the various authorities have sought to meet either with varying degrees of theoretical minutiæ, or by means of working factors sufficiently comprehensive to cover all cases. Some of the formulas take into consideration the compressibility and weights of pile, others neglect one or both. It is easy, therefore, to see that they must differ. None regard the compressibility of the soil as a necessary element in the calculation, nor introduce specific factors of mechanical friction and air resistances as applied to the falling ram. The proper weight-ratio of pile and hammer to secure the best results has received theo-

retical consideration, but as it would be extremely inconvenient and in fact impracticable to select piles of definite weight, it is rightly regarded as a matter to be considered generally. The ram should always be at least as heavy as the pile; generally in practice it is from 25 to 200 per cent. heavier than piles which may be used in the same work.

It is evident further that, however interesting and involved the theoretical study of this question may be, a working formula should contain only those essential factors which can be readily obtained with reasonable correctness in the case of any pile, and it is not worth while nor sensible to waste time in refining a result beyond the refinement of the data. The frictional and air resistances can, however, be experimentally rated by chronographic methods once for all in the case of any particular machine, if that refinement be desirable, and expressed as a decrement for the actual fall. The Trautwine formula, which is probably used more extensively than any other in this country, takes no account of compressibility, weight of pile, or directly of frictional resistance to fall of ram, but considers the work done by the ram to vary as the cube root of the fall. While there is justification for this assumption in dealing with average falls, based upon increased air resistance and greater impact losses, yet the allowance would appear to be too great, giving too low values for the higher falls, and excessive results for very low ones.

In most situations it is necessary to treat piles individually in order that all in any group may be given the same measure of stability under varying conditions observed in driving, so that it is essential to apply the formula to every pile. On this account the form of a working formula is a matter of great importance, and to insure rapid and correct results it must be a simple expression involving only such data as can be quickly and correctly obtained for each pile, and requiring the minimum of computation for instantaneous application.

The only data that can be readily obtained are the weight (w) and the fall (h) of ram, and the penetration (s). The weight of the pile and its compressibility cannot be conveniently determined, and are so modified often by considerations of specific gravity and resilience that they would be of no practical account. In addition to that any "brooming" of the pile-head acts as a cushion, to an extent which completely discounts the theoretical natural compressibility. The consideration of brooming, like other mechanical questions which present themselves

in connection with the art of pile-driving, does not properly come within the scope of this paper. A very instructive and precise record of the enormous cushioning effects due to this cause, as observed in a number of piles driven by a Nasmyth steam hammer, is contained in a very valuable paper on steam pile-driving, by D. J. Whittemore, Past President Am. Soc. C. E.*

If the proposition be accepted that we may profitably discard all working formulas, excepting those that involve only correct and accessible data, we may select from the foregoing list for further comparison the three following typical forms, rejecting the others as being either unnecessarily refined and unsuitable, or inconveniently cumbrous. In all three the notation is as follows:

w = weight of ram in pounds. h = height of fall in feet. s = penetration under last blow, in inches. F = working factor.

1. Weisbach.
Load in pounds.
$L = F \dfrac{wh \times 12}{s}$
$F = \frac{1}{3}$ to $\frac{1}{6}$.

The form of Weisbach, wherein compressibility and weight of pile and elasticity of ram are neglected; this is usually known in America as Sanders'.

2. Trautwine.
Load in tons.
$L = F \dfrac{\sqrt[3]{h} \times w .023}{s + 1}$
$F = \frac{1}{3}$ to $\frac{1}{12}$.

The form of Trautwine, wherein the pile is assumed to "sink appreciably, say, a few inches," under the last blow. In the later editions of this authority, the other form, for smaller penetrations, has been dropped and this is made general. See "Trautwine's Engineer's Pocket Book," 1889.

3. *Engineering News.*
Load in pounds.
$L = F \dfrac{w \times 2h}{s + (c = 1)}$
F = unity.

This name has been given to this form to distinguish it, the name of its originator not being known to the author; it first appeared in the *Engineering News.*

It will be noted that Nos. 1 and 3 are alike in the respect that each gives similar results for equal ram-energies, expressed in foot-pounds, irrespective of the fall, whereas No. 2 introduces a theoretically illogical variation in this product, which, together with the practical inconvenience of using the cube root in rapid application, without any gain in

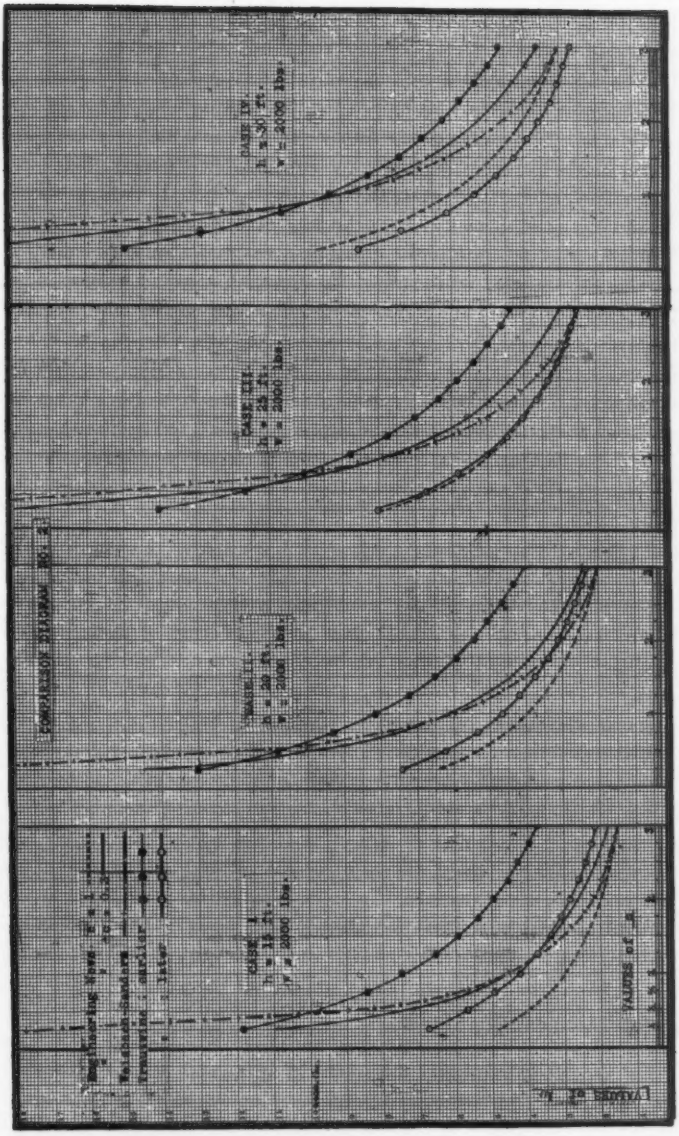
* The Nasmyth Pile-Driver. *Transactions*, Vol. XII, page 441. 1880.

accuracy, would give preference, as far as form alone goes, to either No. 1 or No. 3 as a working formula.

In order to compare readily the results obtained under a variety of conditions, Diagram No. 2 has been prepared, exhibiting the respective values of safe static loads as given by the three formulas, in four cases of varying height of fall, with penetrations ranging from one-quarter inch to 3 inches under constant weight of ram. Were it not for the presence of the cube root in the Trautwine formula, one case instead of four would have been sufficient for comparing the three formulas. It is interesting to trace the effect of this factor through the successive plottings and to note that it is by no means inconsiderable, even within the comparatively limited range of fall.

The distinctive curves are plotted to abscissas of penetration in inches (values of s) and ordinates of working loads (values of L). As the Trautwine formula in its earlier form contained a somewhat higher coefficient and a maximum factor of one-half instead of one-third, two Trautwine curves are necessary to make the comparison comprehensive, the more modern form giving the lower values. In order to show graphically the effect of the constant "c" in the *Engineering News* formula and to prepare for reference further on to a suggested modification of this formula, the curve, shown by the full line of the equation $L = \frac{w \times 2h}{s + (c = 0.3)}$ has been introduced. In the Weisbach-Sanders formula F is taken at one-eighth, and in all four cases w is taken at 2 000 pounds, h being 15, 20, 25 and 30 feet respectively. Values of L are given in pounds, to the nearest thousand.

It is hardly necessary to mention that these curves are merely graphic representations having no generic value. But their comparison is instructive and we are struck by two practical considerations: first, that where the values of s are from one-half inch to 1 inch there is a closer approach to uniformity in the formulas than when s is either greater or less; second, the *Engineering News'* functions with constant ($c = 1$) give values more and more conservative, comparatively, as the values of s become smaller, whereas the curve with c taken at 0.3, while growing conservative as s becomes greater, maintains more nearly an average of the other two where s is smaller. The first observation implies that the experiments preceding the adoption of all three formulas were conducted within a comparatively narrow range of falls; while


CASE IV.
h = 30 ft.
P = 2000 lbs.
CASE III.
h = 24 ft.
P = 2000 lbs.
CASE II.
h = 20 ft.
P = 2000 lbs.
CASE I.
h = 16 ft.
P = 2000 lbs.
Values of s

the second points to the use of the lower value of c as being more logical, for it will be admitted that, other things being the same, the smaller the penetration, the greater the expectation of stability of the support to the pile, as well as the initial sustaining power of the pile itself. Proceeding a step further we find that the c curves converge in the direction of greater penetration, but preserve their conservative quality; whereas, in the opposite direction the Sanders curve gradually passes entirely out of the field of practicability. The conclusion seems to be warranted, therefore, that with a suitable value for c, the *Engineering News*' formula is more reliable and rather more convenient in practical use than Trautwine's, and equally as convenient and more conservative than the Weisbach-Sanders. It is to be borne in mind that within the range of moderate penetration (2 inches or less) ample experience has demonstrated that for ordinary conditions any one of the three gives safe results.

In the discussion of the third division of our subject, the consistent application of the formula to varying conditions, the value of c will be considered.

We have seen that the form of formula No. 3, $L = \frac{2\,wh}{s + c}$ is the simplest, and that its results are reliable and conservative. If, then, it is found to be applicable to all cases without change of form, and, in addition, to be readily adaptable to classification, it will be suitable as a basis of uniform practice and also as a means hereafter of increasing our practical knowledge of the efficiency of bearing piles, by accumulating comparable results. The writer offers, as the only feature in this paper for which originality is claimed, a development of the formula which lends itself in a very elastic and consistent manner to the desired end. It consists simply in making c a tangible variable for natural conditions, and a selective variable within limits according to the requirements of the case. In order to make this perfectly clear we should consider that there are three general cases in which modification may be necessary:

1. Where the load is static and known, but the natural support for the piles is insecure or doubtful.

2. Where the support is reliable, but the load is dynamic.

3. Where the support is not reliable and the load is dynamic.

In the first case, the variation can generally, though not always, be

gauged by the resistance of the material into which the pile is driven, under what may be termed a standard blow. The writer considers that a 2 000-pound ram falling 20 feet, or an equivalent product of 40 000 foot-pounds on a 12-inch pile, constitutes a convenient standard for this purpose, and when the standard blow, or a series of such blows, is delivered on any pile, the observed penetration should in a majority of cases afford a tangible measure of the security of the stratum. If, then, we substitute for the constant value of c a series of values based on the above considerations, we can readily determine with practical correctness how to deal with pile-beds of various degrees of firmness.

The rule adopted by the author is to make c (for static loads) $= .1 + n$ wherein .1 is an arbitrary constant, introduced to prevent exaggerated values of L with very small values of s, and n is made equal in the case of each pile to half the square root of the maximum penetration of the pile under a blow of 40 000 foot-pounds or more.

Table No. 1 on page 110 gives corresponding values of n for different penetrations, and shows that within the extremes of penetration of about one-sixth of an inch to 3 inches under the standard blow the values of c vary from .3 to 1, these being the values for the two curves of this formula in Diagram 2, which we have considered.

In the second case the selective variable should be used, and here the engineer must be guided by his experience, or the experience of others, as to the value of n' to be selected ; this will be further considered.

In the third case both the tangible and selective variables of n and n' must be introduced.

The extreme case in which this formula should be used under the most favorable circumstances, is for a penetration of one-eighth of an inch, as it is difficult to measure smaller penetrations with sufficient accuracy. The maximum value of L, therefore, (using $c = .275$, as given in the table) for $s = 0 +$ would be $L = \frac{2\ wh}{.275} = 7.3\ wh$, which as h is expressed in feet and s in inches, gives a working factor still of $\frac{12}{7.3}$ $= 1.7$. But we should attach little value to small sets, which must always be viewed with suspicion, and often, moreover, indicate results beyond the crushing strength of the pile.

TABLE No. 1.

A General Pile-Driving Formula.

For Static Loads:

L = the working load $= \dfrac{2\,wh}{s + .1 + n}$; w is weight of ram; h is virtual fall in feet; s is set in inches under last blow.

For Dynamic or Vibratory Loads:

$$L = \frac{2\,wh}{s + .1 + n + n'}.$$

n is a function of the observed penetration, s', in inches, under a standard blow of 40 000 foot pounds $= \dfrac{\sqrt{s'}}{2}$.

n' is an arbitrary variable which is dependent upon the character of the pile duty.

Values of n.		Values of n' according to duty classification.	
s'.	$\left\lfloor \frac{s}{2} \right\rfloor_{cs} = n$	n'.	Classification.
$\frac{1}{4}$ inches.	.175	.1	Large buildings to contain light machinery in motion.
$\frac{1}{2}$ "	.250	.2	Long span bridge abutments for railways.
$\frac{3}{4}$ "	.354	.3	" " " " " highways.
1 "	.433	.4	Buildings to contain heavy machinery in motion.
1	.500	.45	Short span bridge abutments and trestles for railways.
$1\frac{1}{4}$ "	.559	.5	" " " " for highways.
$1\frac{1}{2}$ "	.613	.55	Buildings subject to extraneous vibration.
$1\frac{3}{4}$ "	.662	.6	Foundations for machinery.
2 "	.707	.7	Elevator towers in ordinary cases.
$2\frac{1}{4}$ "	.750	.75	Bridge piers exposed to current vibration.
$2\frac{1}{2}$ "	.791	.8	Light houses exposed to ordinary wave action.
$2\frac{3}{4}$ "	.830	.9	Foundations for turn-tables.
3 "	.866	.95	" " pivot bridges.
$3\frac{1}{4}$ "	.900	1.	Chimney stacks exposed to winds.
$3\frac{1}{2}$ "	.936		
$3\frac{3}{4}$ "	.965		
4 "	1.		

This table applies solely to bearing piles; the results are to be taken only when found not to exceed safety as against fiber-crushing, or as against buckling in cases wherein the pile acts, wholly or in part, as a column.

In this consideration the author has purposely confined himself to piles proper and has not extended it to pillars or columns, which they sometimes become when driven through a soft stratum to an unyielding bottom, or into a denser lower stratum. In all such cases great care and discrimination is necessary as to what extent results given by the pile formulas can be used, and also as to the limitation of

driving. These formulas are only for determining the efficiency of piles to distribute, throughout the stratum, loads which its surface cannot sustain. It is quite possible to overload the stratum by driving piles too thickly; it is sometimes thought that if three piles, for instance, are good in a certain area, six will "make assurance doubly sure"; this policy is not only costly and foolish, but in many cases it is dangerous. A certain public building in New Orleans, of which the author has heard, stands on an enormous number of square piles driven so as to touch one another; in other words, there is a solid wall of wood, and instead of each pile having four surfaces in contact with the sustaining stratum, only the outer rows have contact, and they but on two surfaces; there are, therefore, from four to six times too many piles used and the support is still much less than could have been obtained.

There seems to be objection in many quarters to piles for foundations of buildings, which is not warranted in the light of their record in engineering structures, and which greater care and closer economy in their use should dispel. As a rule, in the author's judgment, their sustaining powers are greatly underrated, and too much importance is attached to small "sets." He has before him the specifications for a large structure to rest on piles, which require that "all piles shall be driven to a half-inch set from a 2 500-pound hammer falling 25 feet," and the plans are drawn with the idea that that degree of firmness in each pile can and will be secured. Possibly experimental driving has already determined that the expected conditions will be found, but it is more than likely that in driving the piles, either considerable latitude will be taken or an excess of length of piles will be used. Sometimes in the search for firm bottom, successive tiers of piles have been driven, one presumably on top of the other, to great depths; whereas, if an equal number of piles, or, better still, if a smaller number, but of larger diameter, had been driven in one tier, much greater actual stability would have been secured with greater economy.

The Selective Values for n'.—We have seen how the relative reliability of a stratum can be introduced into the working formula, and how, if desired, the virtual height of fall of ram, to eliminate frictional and air resistances, can be determined. So that for static loads we can with the formula $L = \frac{2\,wh}{s + 0.1 + n}$ readily obtain consistent results.

It now remains to consider what is the nature of the selective process by which individual engineers can judiciously apply the formula to dynamic loads in the great variety of conditions likely to arise, in order, first, that actual security can be attained economically; and, secondly, that the results may harmonize and be comparable. As n increases, L decreases, and if we can construct some convenient arbitrary scale of increments, n', and group upon it the different kinds of service to which piles may be put, we shall approximate very closely to the requirements in any particular case. Owing to the fact that few records of actual performances of piles under varying conditions are at present available it would be impossible to make the scale exact, even if it were theoretically desirable to do so, but the extreme cases can be referred to a conservative basis, and a reasonably accurate subdivision be made. Such a scale of increments, n', to be added to values of n previously determined for static loads, according to exigency, is given in the table. Exception may, perhaps, be taken to the order of arrangement of the groups, which is here offered as an illustration of its application, and which is further exemplified by the working out of two typical cases a and b. We will assume, in both, the values $w = 2\,000$; $h = 30$ feet; $s = \frac{1}{4}$ inch. For the case a we assume s' the penetration under the standard blow to have been one-half inch, indicating a reliable footing, while in case b, $s' = 4$ inches; supplying, from the table, the appropriate values of n and n' we obtain working loads as follows:

TABLE No. 2.

	Case *a*.	Working factor.*	Case *b*.	Working factor.*
Static load	125 940 lbs.	$\frac{1}{11}$	75 000 lbs.	$\frac{1}{19}$
Large building, light machinery.	113 960 "	$\frac{1}{12}$	70 588 "	$\frac{1}{20}$
Chimney stack	61 444 "	$\frac{1}{23}$	46 154 "	$\frac{1}{31}$

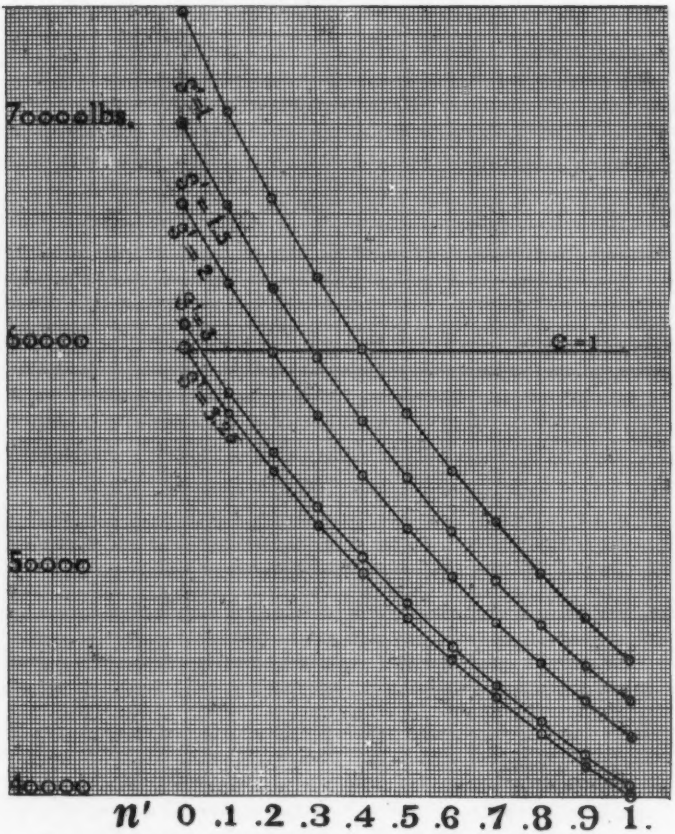
By the *Engineering News* formula, unmodified, the working load would be the same throughout, viz.: $L = \frac{2 \times 2\,000 \times 30}{\frac{1}{2} + 1} = 80\,000$ pounds.

It is, of course, to be understood that the chimney stack foundation is assumed in the premises to be properly designed to meet conditions

* The working factors are deduced, not determined.

COMPARISON DIAGRAM No. 3.

SHOWING THE EFFECT OF THE SUBSTITUTION OF $(.1 + n + n')$ FOR $(c = 1)$ IN THE *Engineering News* FORMULA.



Note.—The case is of a 2 000-pound ram falling 30 feet and giving a set of 1 inch; the horizontal line represents the working load by the formula $L = \frac{2\ wh}{s+1}$ while the curved lines give working loads for various classes of service deduced from application of Table No. 1; each curve indicates a particular value of n belonging to an observed penetration (s') under *standard blow* of 40 000 nominal foot-pounds; for values of s' greater than 3½ inches the modification gives smaller loads, for all classes of service, than the parent formula.

of unequal loading due to wind pressure; the additional security given by the table is against wind vibration.

While there are, presumably, cases of unusual importance or difficulty arising in general practice, which would call for greater increments of n and n' than Table No. 1 affords, yet it is believed that for usual conditions it will be found entirely suitable and consistent with security and economy. The use of larger (or smaller) values of n and n' would, if analogous, extend the capacity of the formula without impairing its usefulness, and while the selection of n' must be made to fit each case, it would be a great advantage to the engineer to be able to select it from comparable quantities, and by precedent rather than through the exercise of his emotions.

It is to be regretted that our members in describing the structures they build say so little about foundations, and it is hoped that in the discussion which this paper may call out, practical examples will be given, taken from actual experience of the behavior of piles in service, under loads of various character, in connection with the rules by which they were driven. Such a list would be of great value, especially if embracing settlements and failures as well as satisfactory performances.

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543.

(Vol. XXVII.—August, 1892.)

THE IRON WHARF AT FORT MONROE, VA.

By JOHN B. DUNCKLEE, M. Am. Soc. C. E.

WITH DISCUSSION.*

General Description.—The iron wharf at Fort Monroe, Va., was built in 1888–89, in accordance with the provisions of Acts of Congress approved August 4th, 1886, and August 10th, 1888. The wharf is located on the site of an old wooden wharf formerly known as the "Baltimore Wharf," and at the terminus of the roadway passing in front of the Hygeia Hotel. The general plan of the wharf is shown on Plate XXII, Fig. 1. It is built on hollow cylindrical cast-iron disc and screw piles (the latter having wooden bearing piles), spaced 14 feet apart from center to center in each direction, and braced by two systems of horizontal bracing of steel and wrought iron. The floor beams, resting on the piles, are steel I-beams, and the floor joist and planking are of pine lumber. A fender system of wooden piles and lumber surrounds the outer faces of the wharf. The floor of the wharf is 7 feet 11 inches above low tide. Its area is about 63 500 square feet.

The wharf extends from the shore in a southerly direction to a depth of 20¼ feet at low tide on the edge of the channel of Hampton Roads. The extreme length of the wharf from the shore to the southerly face

* See page 129.

is 332 feet. The width at the shore and for a distance of 42 feet out from the shore is 56 feet, increasing to 258 feet at a distance of 154 feet from the shore. The outer corners are cut off diagonally, as shown on the plan, so that the width along the south or channel front is 154 feet. These diagonals have a frontage of 59 feet. The corners were planned in this manner at the request of the captains of all the steamboats stopping at Fort Monroe, in order to facilitate the landing of boats during adverse conditions of the wind and tide. It not infrequently happens that the tidal currents in the channel and near the shore are running in opposite directions, while the sub-surface currents are sometimes contrary to those at the surface. The wharf is exposed to northeasterly storms, and all of these conditions combined often make the landing extremely difficult. The bottom of the bay at this point is sand, which is not moved by the ordinary tidal currents, but which shifts under the action of severe easterly storms, so that variations of depth of from 1 to 2 feet resulting from such storms are not unusual.

Cast-Iron Piles.—It was at first proposed to build the wharf on wrought-iron piles with cast-iron discs, similar to those used in the construction of the Coney Island and other ocean piers along the coast. Upon making borings at the site, however, it was found that near the outer limit of the wharf area, in depths of 20 feet at low tide, the stratum of sand was but 6 feet in thickness, while below this, mud was found to a depth of 50 feet or more below low tide. While the thickness of the stratum of sand gradually increases as the water shoals, it was not regarded as affording a sufficient or safe bearing for disc piles, outside the 10-foot curve, particularly in view of the mud beneath and the shifting bottom above.

It was therefore decided to use in depths of more than 10 feet a hollow screw pile resting on and encasing a wooden bearing pile, previously driven and cut off from 3 to 4 feet above the bottom. Owing to the required shape of the iron piles the use of wrought iron was impracticable, and cast iron was therefore adopted as the material of construction. In depths of 10 feet or less a cast-iron disc pile was used. The cast iron was a close-grained, hard, white metal, intended to be without uncombined carbon, and of a character which would best resist the action of salt water, not excessively brittle, and which would satisfactorily bear drilling. All castings were coated inside and outside

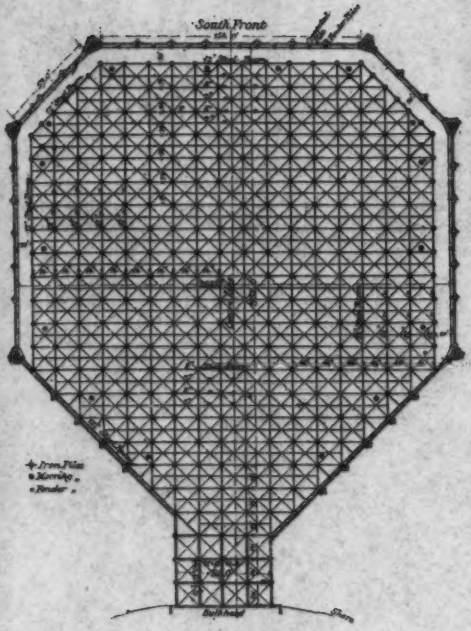
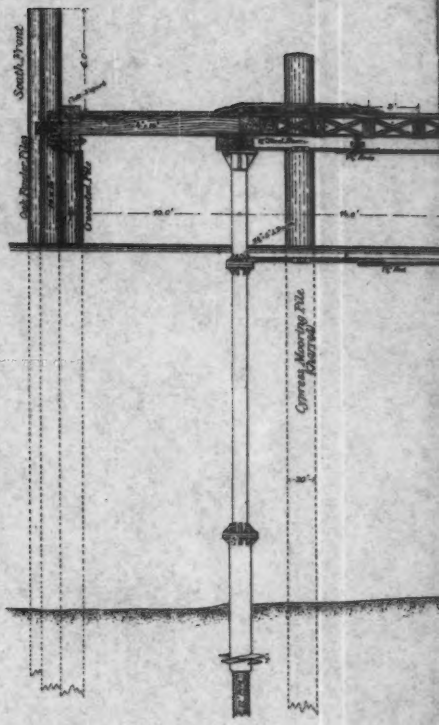


FIG. 1
General Plan
Planking & Floorjoists removed
Scale



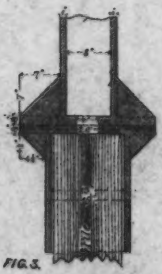


FIG. 3.

FIG. 4
Plan of Flange

FIG. 5.
Plan of Flange, looking up

Screw Pile

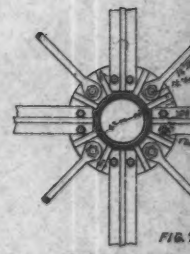
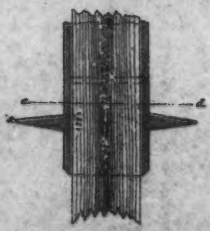


FIG.
With lower bracing



Vert. Section on 'a-b'

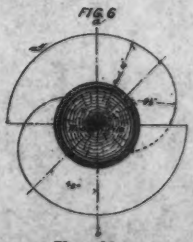


FIG. 6
Plan of Screw
(Section on c-d, Fig. 3.)

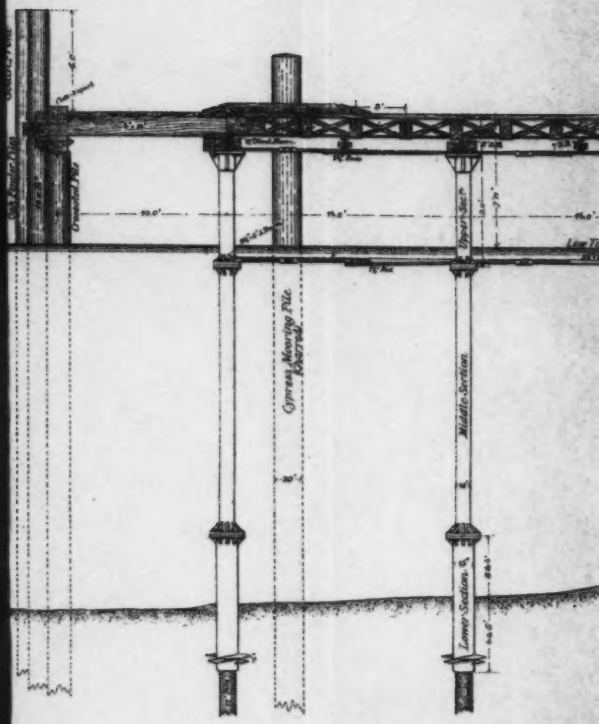
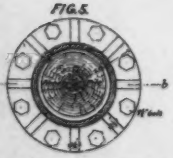


FIG. 2.
Vertical Se
along Centre
Scale

FIG. 5.



Plan of Flange, looking up

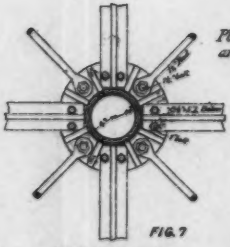


FIG. 7
With lower bracing

Plans of Flange for
and middle Sections

Without

FIG. 6

Scale of Details

Screw
Fig. 5)

PLATE XXII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXVII. Nº 543.
DUNCKLEE ON IRON WHARF AT
FORT MONROE, VA.

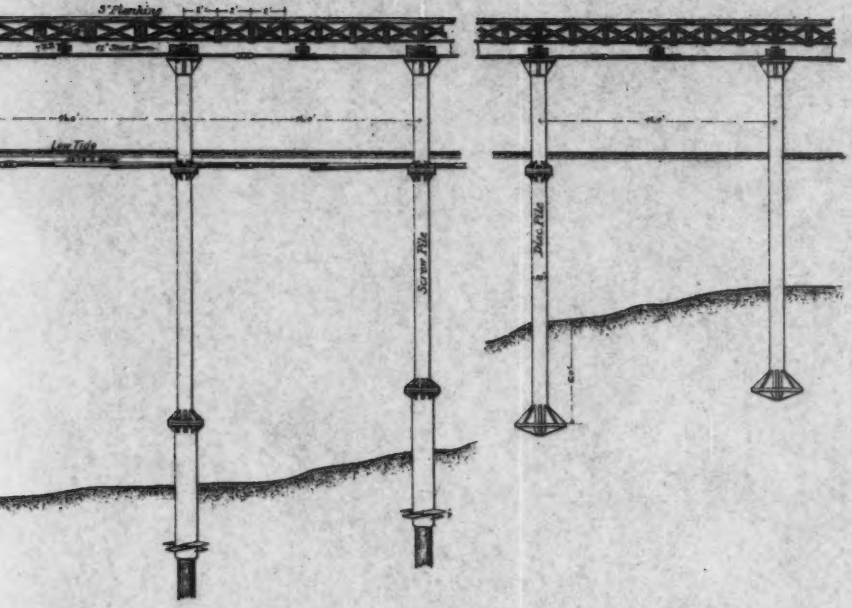


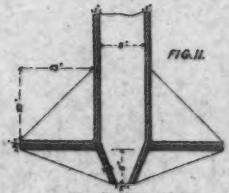
FIG. 2.
ertical Section
along Centre Line
Scale

f Flange for Upper
ddle Sections

Without lower bracing
FIG. 8.

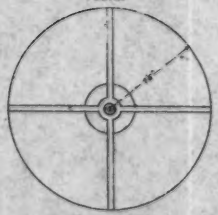
Details

Disc Pile



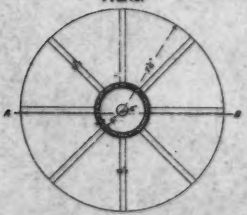
Vert. Section on A-B.

FIG. 9.



Plan - looking up.

FIG. 10.



Top Plan of Disc

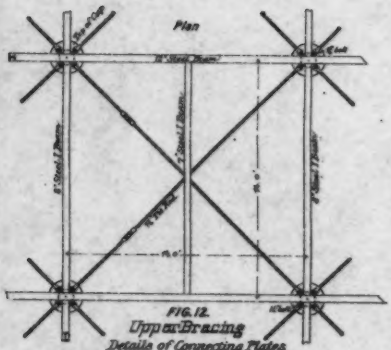


FIG. 12.
Upper Bracing
Details of Connecting Plates omitted.

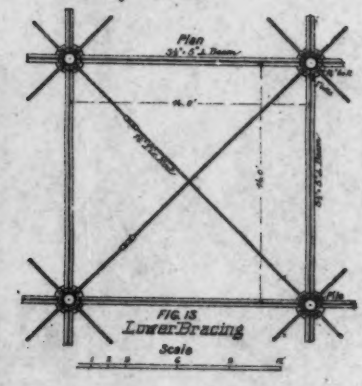


FIG. 13.
Lower Bracing
Scale

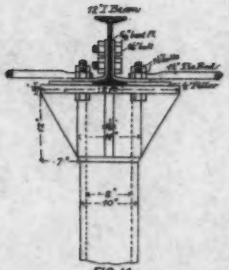
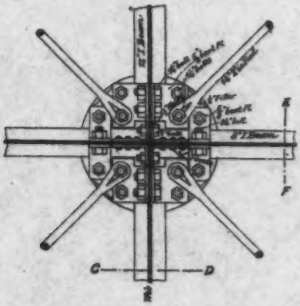
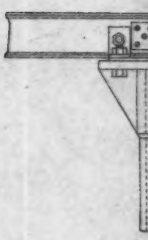


FIG. 14.
Elevation on C-D.
Bent plates securing Floor Beam & Tie Rods.



Plan of Cap & Upper Bracing
FIG. 15



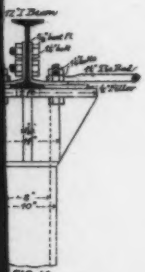


FIG. 14.
...ion on C-D
...securing Floor Beam,
...de Rods.

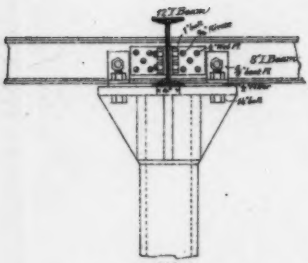
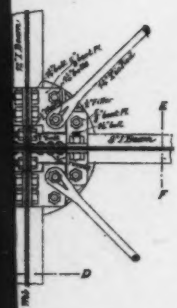


FIG. 16.
Elevation on C-D
Bent plates securing floorbeam & Rods are omitted



...o & Upper Bracing
...G. 15

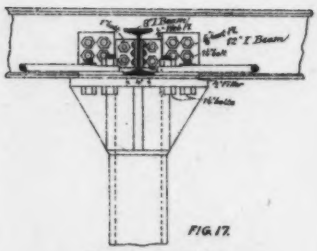


FIG. 17.
Elevation on E-E
Bent plates securing 8" beam omitted

Cap and Upper Bracing

Scale

PLATE XXIII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXVII. Nº 543.
DUNCKLEE ON IRON WHARF AT
FORT MONROE, VA.

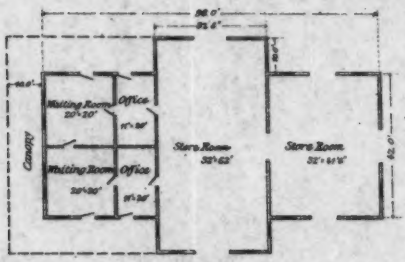


FIG. 19
Plan of Wharf Building
Scale

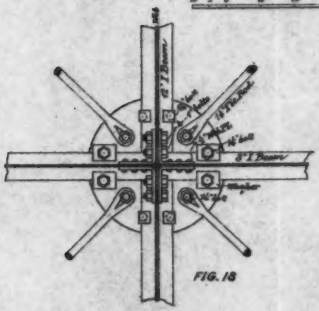


FIG. 18

Plan of connection of Floor Beams with Cap of Pile where Steel Beams were substituted for Oak Beams.

with coal pitch and oil, according to Dr. Smith's process as used for water pipe.

Disc Piles.—The disc piles are hollow cylinders of cast iron, with an inside diameter of 8 inches, and an outside diameter of 10 inches, the iron being 1 inch in thickness. The form and dimensions of the pile are shown on Plate XXII, Figs. 2, 9, 10 and 11. At the foot of each pile is a disc 3 feet in diameter and 1.5 inches in thickness, cast with and forming a part of the pile, the distribution of weight from the pile to the disc being secured by means of eight brackets or ribs, 1 inch in thickness, as shown on the drawing. The disc or base of the pile is provided with a 2-inch nozzle, as shown for the water-jet, the nozzle being cast with four 1-inch ribs bracing it to the under side of the disc. At the top of the pile is a circular flange or cap 2 feet in diameter and 1.5 inches in thickness, cast with four ribs or brackets 1.5 inches thick on the under side as shown on Plate XXII, Fig. 2, and Plate XXIII, Figs. 14, 15, 16 and 17. The thickness of the iron of the pile is made 1.5 inches for a distance of 12 inches below this top flange or cap. To this cap the floor beams are secured and also the upper bracing as hereafter described. Piles intended for depths of from 7 to 10 feet, and which, if cast in one piece, would exceed 20 feet in length, were cast in two sections with flanges 20 inches in exterior diameter and 1.5 inches thick at the point of junction. Each flange has a circular opening through the center, 8 inches in diameter (this being a continuation of the inner surface of the pile cylinder) and is provided with eight brackets 1-inch in thickness, as shown on Plate XXII, Figs. 7 and 8. Bolt holes were drilled through the flanges as there shown. The abutting surfaces of the flanges were planed and the two sections bolted together with wrought-iron bolts, a sufficient quantity of red lead being used to make a tight joint. Eight 1¼-inch bolts were used in bolting the sections together, except where lower bracing was attached, in which case four 1¼-inch bolts and eight 1-inch bolts were used as indicated in Plate XXII, Fig. 7.

In the center of the top flange or cap of each two-section disc pile there is a circular opening 4 inches in diameter. In the one-section disc piles this opening is made 8 inches in diameter, to permit the removal of the core bar. The disc piles were sunk by means of the water-jet without special difficulty, except where logs or piles were encountered. The discs are about 6 feet in the sand.

Screw Piles.—The screw piles are of the form shown in Plate XXII, Fig. 2, and were generally cast in three sections designated as the lower, middle and upper sections. The lower section, which rests on and encases the wooden bearing pile, is 8 feet long and 13 inches in interior diameter, the iron being 1 inch in thickness. At the top of this section is a flange 24 inches in exterior diameter and 1.5 inches thick, provided with eight supporting brackets each 1.5 inches thick. In the center of this flange is a 4-inch circular opening. About 1 foot above the lower end of this section of pile are two screw-pile blades, with a maximum diameter of 32 inches. The blades are 2.5 inches thick at the junction with the cylindrical pile, this thickness diminishing to three-eighths of an inch at the edge of the blade. The details of the lower section are shown on Plate XXII, Figs. 3 to 6 inclusive. The middle section of the screw pile is 8 inches in interior diameter and 10 inches in exterior diameter, the iron being 1 inch in thickness. The lower flange is 24 inches in diameter and 1.5 inches in thickness and is provided with eight brackets each 1.5 inches in thickness. There is a circular opening 4 inches in diameter in the center of this flange, corresponding to the opening in the upper flange of the lower section of the pile. The upper end of the middle section is provided with a circular flange 20 inches in exterior diameter and 1.5 inches in thickness, provided with eight brackets 1 inch in thickness, as shown on Plate XXII, Figs. 7 and 8. This flange is cast with an opening in the center of the full inner diameter of the pile (8 inches). The variations in the depth of water were provided for by varying the lengths of the middle section by multiples of 1 foot, the lengths ranging from 7 to 17 feet. The upper section of the pile is also 8 inches in interior and 10 inches in exterior diameter, with a flange on the lower end 20 inches in exterior and 8 inches in interior diameter and 1.5 inches in thickness. On the four outer rows of piles on the sides and six outer rows on the south end of the wharf, the lower system of bracing is attached to this flange, and the brackets are spaced and arranged so as to facilitate this connection, as shown on Plate XXII, Fig. 7. On the piles comprising the interior rows, the brackets are equidistant, as shown on Plate XXII, Fig. 8. The top of the upper section has a flange or cap 24 inches in diameter and 1.5 inches in thickness to receive the steel floor beams and upper bracing. This flange is supported on the lower side by four brackets 1.5 inches in thickness, extending to a point 12

inches below the flange, the thickness of the iron of the pile being increased in this space to 1.5 inches. There is a circular opening 4 inches in diameter through the top of the cap. The details of the cap are shown on Plate XXIII, Figs. 14, 15, 16 and 17.

The upper sections are all 6 feet 9 inches long. On piles other than those to which the lower system of bracing is attached, the upper and middle sections are cast in one piece, where the length of such combined section does not exceed 20 feet. The connecting flanges are of course omitted in this case.

All flanges were planed on their abutting faces, and all holes were drilled. A sufficient quantity of red lead to make a tight joint was spread on the abutting faces of the flanges before bolting them together. The lower and middle sections were bolted together with eight 1.5-inch bolts with nuts, and the middle and upper sections with four 1¼-inch bolts and eight 1-inch bolts, where lower bracing was attached, and with eight 1¼-inch bolts where there was no lower bracing. All bolts were painted with two coats of iron-ore paint before being used.

The wooden bearing piles were of pine, creosoted with 12 pounds of coal-tar creosoting oil to the cubic foot. The piles were 10 inches in diameter at the smaller end, and the portion of the pile to be encased by the lower section of the iron screw pile was worked down so as to be 12 inches in diameter and perfectly straight. The length of pile to be so covered after the pile had been driven and cut off was 8 feet; but as it could not be known just where the pile would bring up, it was necessary to dress down a much greater length of the pile. In order that the iron superstructure should come together, it was necessary that the piles should be driven exactly 14 feet from center to center, and perfectly true to line. The use of a floating pile-driver was therefore inadmissible, and a land pile-driver was used, supported on a false work of temporary piles and stringers, the temporary piles being afterward withdrawn. The bearing piles were driven to a refusal equivalent to 1 inch for a hammer weighing 2 200 pounds and falling 15 feet. The length of pile required was generally from 50 to 60 feet, and the length of pile in the ground ranged usually from 20 to 30 feet. Each pile was cut off by a circular saw at the proper depth to receive the cast-iron pile, this being 4 feet more or less (usually less) above the bottom of the bay. The sections composing the iron screw pile having been bolted together, the iron pile was carefully lowered and set

in place over the wooden pile, until the lower or screw section rested on the sand. Wooden levers were then attached to the cap and the pile screwed down into the sand until the flange at the top of the lower section came to a firm bearing on the top of the wooden pile. It was found that during this operation the sand worked up between the wooden pile and the inside of the iron pile, and, accumulating at the top of the wooden pile, prevented the flange from reaching its true bearing. In order to obviate this difficulty, the water-jet was employed to force out the sand.

The hollow iron piles were afterward filled with concrete composed of one part of Portland cement, two parts of sand and three parts of pebbles. The opening through the lower flange was first closed with neat Portland cement placed in small bags and forced down so as to cover the hole. After the cement paste had set, the water was pumped out of the interior of the pile and the concrete put in. The concrete was intended especially to prevent the corrosion of the interior of the pile by salt water.

Floor Beams.—The floor beams, except as noted below, are 12-inch steel I-beams, weighing 56.5 pounds to the foot, in lengths ranging usually from 14 to 28 feet. The beams are laid parallel to the longitudinal center line of the wharf in order that, acting as struts, they might better resist and distribute the shocks coming against the outer face of the wharf, which it was assumed would be the greatest to be resisted. When the plans of the wharf were first prepared, it was proposed to use 12 x 14-inch oak floor beams with 12 x 12-inch oak transverse struts on the inner part of the wharf, for a distance of 126 feet from the shore, the available funds being insufficient for iron or steel beams over the entire wharf area. While the work was in progress, however, an additional appropriation was made by Congress, and it was then decided to use steel beams on the entire wharf except for a distance of 42 feet from the shore, where the oak beams in place were retained. Meanwhile the caps of the piles to be used within the above-mentioned area had been drilled for the connection of oak beams. In order to utilize these piles the connection of the steel beams was adapted to the caps as drilled and consisted of four 1¼-inch bolts with nuts passing through the flange of the I-beam and the cap of the pile, as shown on Plate XXIII, Fig. 18.

Beyond the distance of 126 feet from the shore the steel beams are

secured to the caps of the piles by means of four five-eighth-inch bent wrought-iron plates, two on each side of the beam. Each pair of plates is secured to the beam by four 1¼-inch bolts passing through the web and each plate secured to the cap by two 1¼-inch bolts, as shown on Plate XXIII, Figs. 14 to 17 inclusive. At joints the abutting beams were connected by two five-eighth-inch fish plates and four 1¼-inch bolts, passing through the webs. All joints come on the centers of piles and, as a rule, adjacent beams break joint by a panel length of 14 feet.

The above is the typical connection. On the sides and near the outer diagonal corners of the wharf, special details for these connections were worked out, and in some cases it was necessary to use piles with caps 30 inches in diameter to obtain space for the secure connection of all the beams and struts meeting on the pile.

Upper Bracing.—The upper system of bracing extends over the entire wharf and is shown on Plate XXII, Fig. 1, and Plate XXIII, Fig. 12. The tops of the piles are connected by 8-inch steel I-beams weighing 30 pounds to the foot laid transversely or in a direction at right angles to the line of the floor beams. These 8-inch beams abut against the webs of the 12-inch floor beams and are secured to the caps of the cast-iron piles by bent five-eighth-inch wrought-iron plates and 1¼-inch bolts with heads and nuts, two such plates being used on each end of a beam, or four to each pile. The connection between the 8 and 12-inch beams is effected by one-half-inch angle plates riveted to the former with three-quarter-inch rivets and bolted to the latter with 1-inch bolts. The four piles forming the corners of a bay or square are connected by two 1¼-inch round wrought-iron diagonal tie rods with eyes, attached to the caps of the piles with 1¼-inch bolts. Each rod has a turnbuckle. Midway of the bay a transverse 7-inch steel I-beam, weighing 25 pounds per foot, connects and gives lateral support to the webs of the floor beams, the connection being similar to that of the 8-inch beams. On the south front and the southeast and southwest diagonal corners of the wharf, 12-inch steel I-beams replace the 8-inch beams and form the iron front of the wharf. In order to resist the severe pressures and shocks coming against these corners additional iron piles have been put in and diagonal 12-inch I-beam struts used in place of the tie rods. As already stated, oak struts are used for two panel lengths from the shore.

The fact that the I-beam is not the best form of strut was fully

recognized in preparing the plans. It was thought, however, that the usual form of strut, two latticed channels, would present such a large surface area to the corrosive action of the salt air and spray as to make its use undesirable. The I-beam was adopted as best meeting all the requirements of the case, both as to limited area of exposed surface and facility of connection with the iron piles and floor beams.

All wrought iron was thoroughly painted with iron-ore paint.

Lower Bracing.—The system of lower bracing is put in on the three outer bays (attaching to four rows of piles) on the east and west sides of the wharf, and on the five outer bays (attaching to six rows of piles) on the south end of the wharf (see Plate XXIII, Fig. 13). The system consists of longitudinal and transverse lines of rolled ⊥-iron struts, weighing 16.15 pounds per foot, and diagonal 1¼-inch round tie rods (with turnbuckles), all bolted to the lower flanges of the upper sections of the piles as shown in Plate XXII, Fig. 7. On the bays near the southeast and southwest diagonal corners, where extra iron piles are placed, diagonal struts are substituted for ties.

Flooring.—The floor joists are of 4 x 14-inch heart pine lumber laid transversely and resting on and notched down 2 inches over the steel floor beams. They are spaced 2 feet apart on centers, and on the outer bents are secured to the floor beams by wrought-iron straps to prevent lifting from the action of the wind and waves during storms. Near the south face of the wharf the joists are 8 x 14 inches and are spaced 3 feet on centers.

The planking is 3-inch heart pine plank laid at right angles to the line of the floor joists, and spiked thereto with 6-inch cut spikes, two at each intersection of a plank with a floor joist. All floor joists are bridged with 2 x 3-inch cross bridging at intervals of 4 feet. The details of the flooring are shown on Plate XXII, Fig. 2.

Fender System.—In order to prevent, as far as possible, injury to the cast-iron piles from the striking of steamers against the wharf while making landings, a wooden fender system or extension, about 10 feet in width, was built around the five most exposed faces of the wharf, shown in cross-section on Plate XXII, Fig. 2. It consists of creosoted pine piles driven about 10 feet apart, on lines 10 feet outside of and parallel to the above faces of the wharf, on which rest 12 x 14-inch oak caps drift-bolted to the piles. Floor joists of 4 x 14-inch pine lumber rest on this cap and extend back to the main wharf, the space being

floored over with 3-inch plank. Directly above the cap a 12 x 14-inch oak back-log is placed, projecting 3 inches above the wharf, bolted down through the cap, as shown, with 1-inch round bolts with nuts. Along the face of the wooden extension are bolted two 10 x 12-inch oak fender timbers, the lower bolted to the cap and the upper to the back log with 1-inch bolts spaced 2 feet apart. Inner fender timbers from 12 x 12 to 12 x 16 inches are bolted to the face of the iron wharf proper, as shown on Plate XXII, Fig. 2. These are intended to protect the iron work in case a steamer cuts its way through the outer fender.

The fender piles are of oak, driven in groups of three piles, at intervals of about 20 feet along the outer front of the fender system, bolted together and to the caps with 1-inch bolts with heads and nuts, and also secured from movement by 8 x 8-inch oak "chocks" (see Plate XXII, Figs. 1 and 2). At the six salient angles of the wharf, larger groups of about fifteen fender piles each were driven and bound together by means of 1-inch galvanized wire rope. These large groups are intended to resist the great pressure resulting from the partial turning of a steamer against the wharf, such a group acting as a fulcrum against which the steamer bears while the bow, or stern, is swung to make a landing or to leave the wharf on any desired course. Sixteen mooring piles are driven at convenient points near the outer edge of the wharf and braced to the wooden floor system. The fender and mooring piles are shown on Plate XXII, Fig. 1.

Bulkhead.—At the shore end of the wharf a bulkhead was built of a continuous row of wooden piles capped with a 12 x 12-inch pine timber, the cap being drift-bolted to each pile by inch square drift bolts, each 2 feet long. The filling behind the bulkhead is sand.

Wharf Building.—After the completion of the wharf proper, a wharf building was constructed of wood, as shown in plan on Plate XXIII, Fig. 19. The length of the building between centers of end posts is 98 feet; the width of the building at the north and south ends, between centers of posts, is 42 feet. These dimensions, it will be observed, are multiples of the panel length (14 feet), and the sills of the north and south wings (each 32 feet 8 inches long) are laid on the center lines of the piles. The middle third of the building (also 32 feet 8 inches long) has two wings facing east and west, each projecting 10 feet. A canopy 10 feet wide extends from these central wings around the south end of the building. This building provides two

store-rooms, one 41 feet 6 inches x 32 feet, and the other 32 feet x 62 feet, two waiting-rooms, each 20 x 20 feet, and two office rooms, each 11 x 20 feet. The framing is of Virginia pine and the roof trusses of Georgia pine, wrought iron being used for tension rods. The exterior (rough) sheathing is of Virginia pine and the siding of cypress. The waiting-rooms and offices are finished in Georgia pine and provided with fire-places.

Materials.—The total quantities of materials, etc., used in the construction of the wharf (not including the building) were as follows:

Cast-iron piles, number	331
Cast-iron piles, weight	1 287 079 pounds.
Wrought iron and steel, weight	749 340 "
Creosoted wooden piles, number	368
Fender piles, number	180
Mooring piles, number	16
Pine lumber	385 793 feet B. M.
Oak lumber	30 124 "
Bulkhead	151½ linear feet.

COST.

The contract prices for the work complete were as follows:

Cast iron, 4½ cents per pound.
Wrought iron and steel, 5 cents per pound.
Pine lumber, $33 per M.
Oak lumber, $42 per M.
Creosoted wooden piles, $22, $24 and $30 per pile.
Fender and mooring pi[illegible]s, $10 per pile.

The cost of the wharf, exclusive of the building, was	$121 306 41
The contract price for the wharf building was	7 439 67
Total cost (exclusive of engineering and contingencies)	$128 746 08

Engineers and Contractors.—The work was under the charge of Lieutenant-Colonel Peter C. Hains, Corps of Engineers, U. S. A. The writer was the principal assistant engineer on this and other works in Colonel Hains' charge. The inspector of construction was Mr. James Ware. The contractors for the wharf proper were the Groton Bridge and Manufacturing Company, of Groton, N. Y. The contractors for the wharf building were Messrs. Darden & Ashe, of Hampton, Va. All plans and detailed drawings were furnished by the engineer.